Engineering News

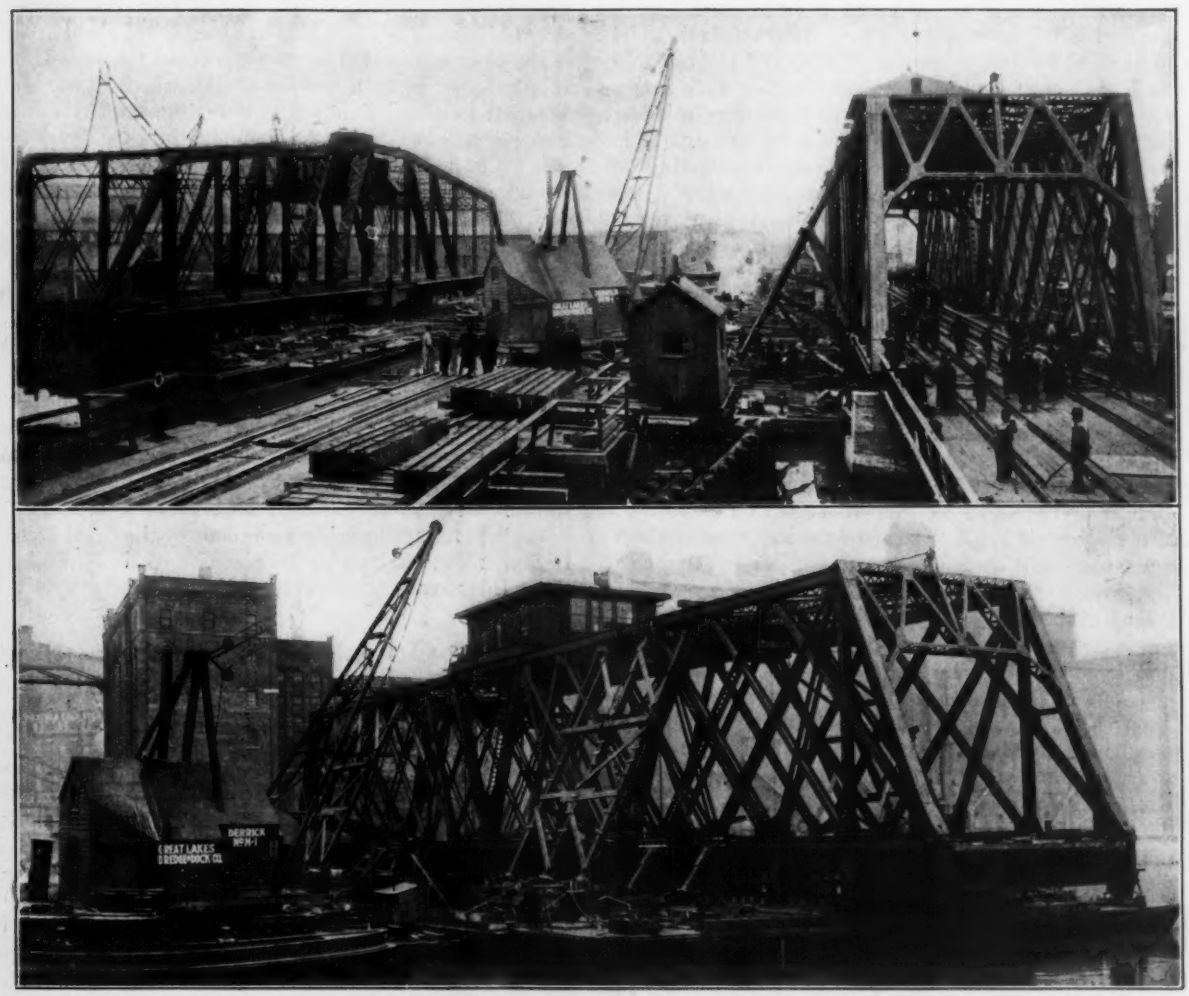
VOLUME 73 APRIL 8, 1915 NUMBER 14

Shifting Two Milwaukee River Drawbridges; C. & N. W. Ry.

For the renewal of its swingbridge over the Milwaukee River at Milwaukee, Wis., the Chicago & North Western Ry. shifted the old bridge about 60 ft. downstream to allow for work on the substructure, and then built the new span on falsework forming an extension of the pile protection on the upstream side of the center pier in the middle of the river. This was described fully in ENGINEERING NEWS, Nov. 26, 1914.

On Mar. 14, 1915, the old drawbridge (500 tons) was floated off its temporary center pier, and the new span (800 tons) was lifted off its falsework upstream, turned around across the channel, floated down and lowered into place on the permanent center pier. All this was done, with time to spare, between the 10:15 a.m. train over the old bridge and the 4 p.m. train over the new bridge in its final position.

The new bridge had been so located for construction that its downstream end was just cleared by the upstream end of the old bridge when the latter was swung open. This permitted cars of material to be run directly onto the rails of the new span. In this way the operating machinery (electric power and gas-engine), operating house, the tracks, guard rails, etc., were all installed in advance, so



FLOATING TWO DRAWBRIDGES ON THE MILWAUKEE RIVER AT MILWAUKEE, WIS.; CHICAGO & NORTHWESTERN RY. (MAR. 14, 1915)

(The upper view shows the new bridge at the right, in its permanent position, but still floating on the scows. At the left is the old span, floated downstream from its position in the temporary line at the middle of the view. This bridge originally occupied the position of the new bridge. The lower view shows the new span floating in the river and shows the system of struts and blocking to distribute the load over the ends of the scows.)

that the new bridge could be swung for navigation purposes as soon as it was installed upon its center pier.

Two openings had been left in the falsework for the two large scows that were intended to lift the bridge. These were the same scows that were employed in lifting the old bridge in October, as already described in ENGINEERING NEWS, and were adequate for lifting the 900 to 1000 tons, including blocking, pumps, etc. But as the bridge during construction was erected 1½ ft. higher than its final position (to avoid too low a cutoff on the falsework piles), the critical feature became the ability of these scows, when at the permanent center pier, to be sunk this additional 1½ ft. and still to have enough buoyancy to carry the entire bridge load absolutely level while the structure was being accurately located upon and lowered to its center pier.

Half the weight of the bridge is to be carried on the center bearing, with the other half on the turntable wheels. As one of the two scows was of steel and the control of such steel craft is difficult when well filled with water, a disastrous accident might result to the bridge in the setting operations should this scow list quickly or sink underneath the bridge. For this reason a wooden coaming 2 ft. high was built around the deck to prevent water flowing over the deck in case the scow tipped. As an additional precaution it was decided to assist the two large scows by removing some of the falsework piles on each side of the openings and placing a small wooden scow alongside each of the larger ones.

Only about 100 tons of bridge load was figured to go upon each of the smaller scows, the two principal scows carrying a load of about 750 tons, including the bridge. The blocking on the scows was arranged to distribute the load, the pumping machinery, etc. It was also thought that the two additional small scows would lend greater stability to the load when carried entirely on the scows, as the bridge stood about 45 ft. above the water and the heavy winds prevalent at this season might cause a tipping of the scows.

Each of the large scows was 100 ft. or more in length. As the bridge, which was only 32 ft. wide, crossed them in the middle, long struts were placed from the top chord to the ends of scows, and inclined posts and A-frames from the lower chord, as shown in the accompanying view. These distributed approximately 60% of the load on each scow away from its middle third, thus producing about the same stresses in the trusses of scows as would be occasioned by the practically uniformly distributed loads (such as sand, stone, gravel, etc.) that they were designed to carry.

A nice adjustment of the wedges under the long struts was necessary in blocking up against the bridge in order to get the proper share of load out to the ends of the scows as pumping out progressed. The deflection of the trusses of the timber scow was compensated by the compression of the long struts. The steel scow being very stiff, figured very little deflection, and the proper share of load could be brought on the ends only by wedging the struts tightly before pumping out started, and also by slacking off the jacks under the bridge at the middle of the scow as the load came on.

On Mar. 13 the weather was favorable, with very little surge in the river, and the new bridge was wedged up against the blocking on the water-filled scows. It was watched all night by crews and warning tugs to protect the outfit from damage by passing boats, and to be ready to knock out the wedges if any considerable change of river level should occur due to shifting of the wind. At daybreak on Mar. 14 pumping out of the scows was commenced by eight pumps, with leads of steam hose from the tugs, and at 7:30 a.m. the bridge had been lifted about 17 in. At 8:30 it had been shifted sideways out of the falsework, taken upstream far enough to turn around in the river and brought down over and across the falsework, the upper timbers of which had been removed meantime to clear the large operating pinions of the bridge.

The floating outfit was tied up within 100 ft. of the permanent location, with men on it to watch the scows, and attention was then turned to the old bridge, by floating under it a pair of water-filled scows in each draw. At 10 a.m. the old bridge was wedged against the blocking on these scows; and at 10:15, when the last train had crossed, the eight pumps on this set of scows were started. At 11:15 a.m. the old bridge had been lifted about 16 in. off its pier and removed downstream out of the way. Later it was towed about a mile to a slip, where it was dismantled in a few days. The new bridge was then set over the permanent pier, and pumping into the scows (to lower it to its center pier) was started at noon. At 12:45 the adjustment of the center bearing was pronounced correct. The wedges were at once knocked out and the scows released soon afterward. This put the new bridge in shape for railway traffic before 2 p.m. Some further work was necessary to complete the adjustments for swinging the drawbridge for navigation purposes, as a section of turntable rack had been left off the center pier at each side of the bridge in order to allow the two large operating pinions attached to the bridge to clear in lowering the bridge. These pinions were quickly meshed in by partial swinging of the bridge by hand, after which these rack sections were placed, and the operating machinery was tested with its two kinds of power.

The removal of the bridges was done by the Great Lakes Dredge & Dock Co., under the direction of W. H. Finley, Chief Engineer of the Chicago & North Western Ry.

Steel Construction for the Schenley Theater, Pittsburgh

By C. N. HAGGART*

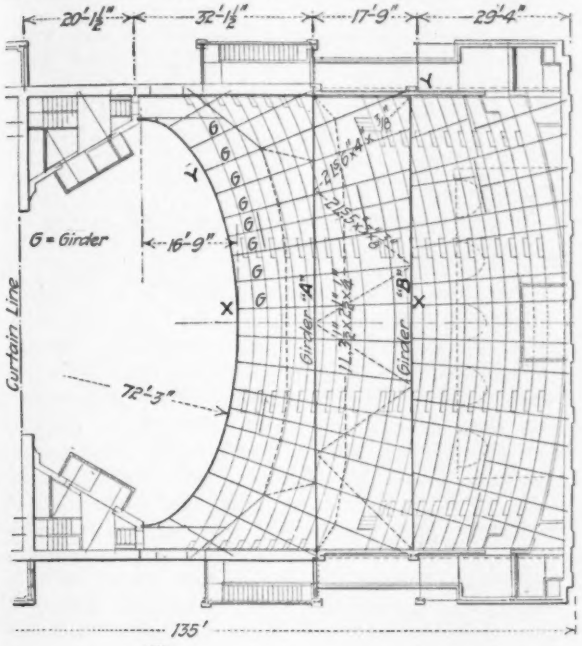
The Schenley Theater, which has been recently constructed in Pittsburgh, Penn., well illustrates in its balcony framing a type which gives a maximum of stiffness and strength with a minimum of shop work.

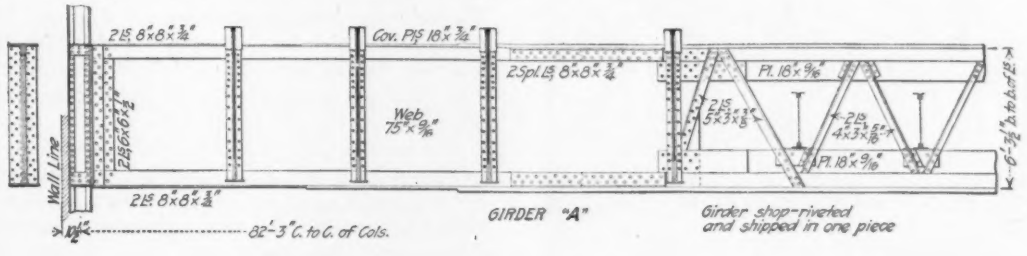
The outside dimensions for the theater building are 100x135 ft. The stage has a proscenium opening 39 ft. wide and 32 ft. 6 in. high. The seating capacity is about 2000. The balcony is constructed with clear span over the main floor. The two main girders, *A* and *B*, have a span of 81 ft. 3 in. out to out of end connecting angles. The forward main girder carrying the cantilever girders is composed of one web 75x$\frac{9}{16}$ in., four angles 8x8x¾ in., and four cover-plates 18x¾ in. The rear main girder is made up of one web 84x$\frac{7}{16}$ in., four angles 8x8x ¾ in., and four cover-plates 18x⅝ in.

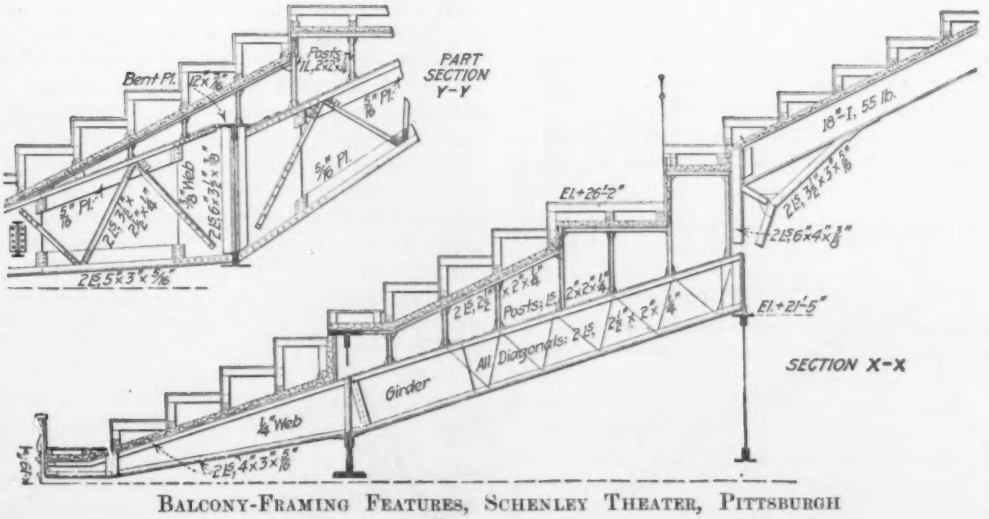
The forward girder (*A*) is made with open web in the middle section, so that the four central cantilever girders can pass through. The other cantilever girders

*Of Haggart Bros., Engineers, 323 Fourth Ave., Pittsburgh, Penn.

frame into girder *A* for its full depth and have a strap over the top to transmit the top-chord tension. Owing to the shape of the balcony there is not enough available depth near the middle of the main girder to accommodate a tension splice over its top flange, whereas the shear being small here allows the use of open-web construction.







BALCONY-FRAMING FEATURES, SCHENLEY THEATER, PITTSBURGH

The balcony framing was designed for a live load of 125 lb. per sq.ft. on the secondary girders and 100 lb. per sq.ft. for the main girders and columns.

The roof framing consists of two main trusses 6 ft. 6 in. deep at the center, two cross-girders 4 ft. 6 in. deep, and 12-in. and 7-in. I's spaced 6 ft. apart for receiving the concrete roof slab. Below, the roof framing carries steel furring for the ceiling, and a light steel-and-concrete walk for access to lights, etc. An elliptical framing at the center for a light dome in the ceiling completes the ceiling and roof framing.

The total weight of steel (stage framing included) was 320 tons; it was fabricated and erected by John Eichleay, Jr., Co. The theater was designed and built under the supervision of H. E. Kennedy Co., architects. The writer's firm was structural engineer, designing the steelwork. The Davis Theater, now under construction in Pittsburgh, is being built to nearly the same structural lines as the Schenley Theater, the designs being made by the same architects and engineers.

Proposed Experiments and Observations on hydraulic and structural engineering problems by the United States Reclamation Service are outlined in the "Reclamation Record" (Washington, D. C.), for March, 1915. The list of subjects for investigation by the engineers connected with the various projects is as follows:

(a) Loss of head through various types of head gate structures.
(b) Loss of head at canal contraction and entrance to flumes, pipes, tunnels, etc.
(c) Loss or gain of head at canal enlargement and outlet of flumes, pipes, tunnels, etc.
(d) Coefficient of discharge of reservoir regulating valves.
(e) Effectiveness of stilling pools.
(f) Surge or fluctuation of head in power-plant pipes.
(g) Effect of curvature or resistance in pipes, flumes and lined channels.
(h) Value of "n" for lined channels, flumes and tunnels.
(i) Depth over spillway lip of siphon spillway to start siphon.
(j) Efficiency of spillway siphons.
(k) Strength of reinforced-concrete pipe.
(l) Coating of metal pipe.
(m) Life of wood pipe and metal flumes.
(n) Seepage losses from canals in various materials.
(o) Seepage and evaporation from reservoirs.
(p) Drainage run-off from seepage of irrigation water.

Rock-Fill Dam with Some Extraordinary Foundation Problems

By M. C. Hinderlider*

SYNOPSIS—The Beaver Park Dam, a rock-fill irrigation dam across a narrow gorge in southern Colorado. As originally designed, the spillway was over an adjoining ridge, which on investigation proved to be of porous formation with bedrock rapidly sloping away from dam. Extraordinary precautions taken to prevent seepage through this formation. Spillway transferred to dam proper.

There has recently been built in the high mountains of southern Colorado a masonry dam of the loose rock-fill type, the final completion of which involved some rather difficult problems not usually encountered in such work. The actual design and construction of the dam involved no unusual features, but conditions affecting not the stability of the structure but endangering its usefulness developed before the completion of the work and made necessary very material changes in the original plans. It is mainly these unusual difficulties and the methods adopted in an attempt to circumvent them which may be of interest and which the writer will attempt to describe in some detail.

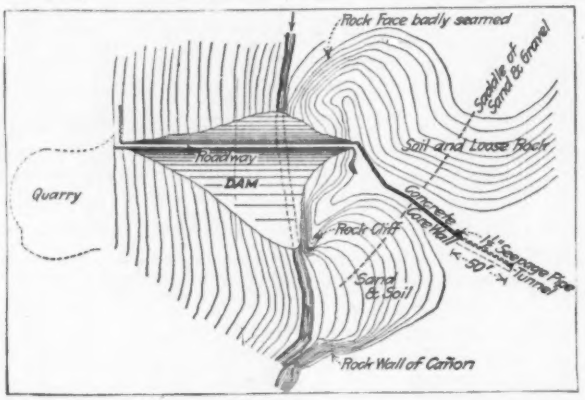


Fig. 1. Plan of Dam Site of Beaver Park Dam, Colorado

*The Field, Fellows & Hinderlider Engineering Co., Denver, Colo.

The structure consists of a loose rock-fill dam with a maximum height of 108 ft., constructed across Beaver Creek, a tributary of the Rio Grande River in Colorado. It was built by the Mosca Irrigation District to store a portion of the flood waters of Beaver Creek for irrigation purposes. The capacity of the reservoir so formed is small, being but 4500 acre-ft., or 196,020,000 cu.ft.

Beaver Creek is a typical mountain stream heading well up on the northwest slopes of the San Juan Mountains, which reach an altitude of about 13,000 ft. above sea level. The drainage basin of about 50 sq.mi. is well timbered with spruce and pine, the open parks being well sodded with a fine growth of bunch grass. Practically the entire drainage basin is located within the Rio Grande National Forest Reserve. The annual run-off from this basin is more than ample for the requirements of the reservoir.

At the dam site (Fig. 1) Beaver Creek Cañon narrows to a gorge of about 60 ft. in width at the bottom and about 200 ft. in width at an elevation of 90 ft. above the creek bed. This constriction in the cañon is formed by the projection of a nose or promontory of rock, extending out from the mountain on the left-hand side of the creek, forming a narrow ridge or saddle, which while sufficiently innocent looking proved to be a most serious obstacle to the early completion of the dam, and continues to present a condition which has caused serious trouble. This promontory may be noted at the left of the view in Fig. 4. The right abutment consists of the sloping wall of the mountain on that side of the cañon.

The basic formation at and in the vicinity of the dam consists of trachite, a soft porous material which very much resembles gray sandstone. This formation is badly broken up and full of seams, and hence forms an unsuitable material upon and against which to rest a structure of such height as the one in question. Such conditions do not necessarily endanger the stability of the dam when properly designed and constructed, but obviously no amount of care exercised in the construction can prevent a large amount of leakage through the porous mountain walls.

The writer had no connection with the work in question until the dam proper was about 75% completed, at which time he was employed as consulting and supervising engineer to carry the work to completion.

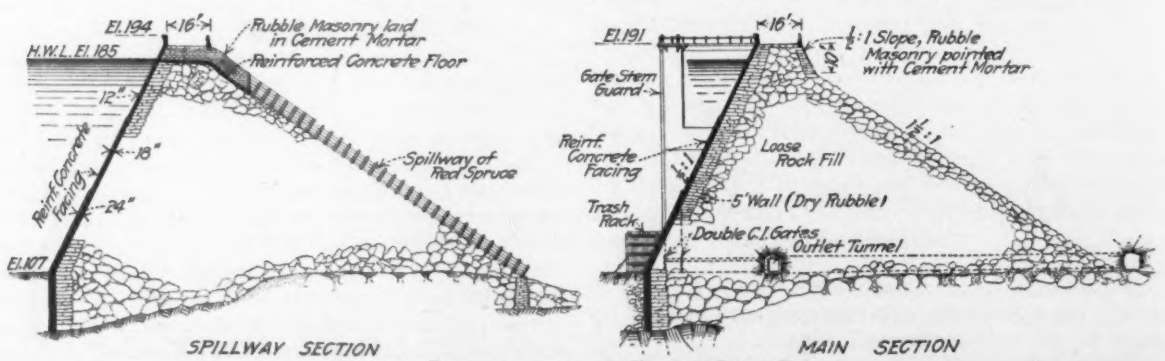


Fig. 2. Sections through Beaver Park Dam

Prior to the initiation of construction work by the District in November, 1912, two or three test pits had been sunk in the bed of the creek and on the right-hand wall of the cañon in an attempt to locate bedrock. These pits were shallow and not of sufficient scope to furnish reliable data. At the left-hand side of the bottom of the cañon, bedrock appeared on the surface, and it was assumed that it would be available within a few feet of the surface the entire distance across the cañon, which later events proved erroneous. It was also the original plan to locate the dam 50 ft. above its present location and to utilize the middle of the narrow ridge or spur, heretofore referred to, for a spillway site. The topographic features were excellent for the purposes in mind, and the surface conditions appeared favorable. No examination of the subsurface conditions beneath this saddle were made preliminary to the location of the dam, and no serious study seems to have been made of the conditions at this point until the writer assumed charge of the work.

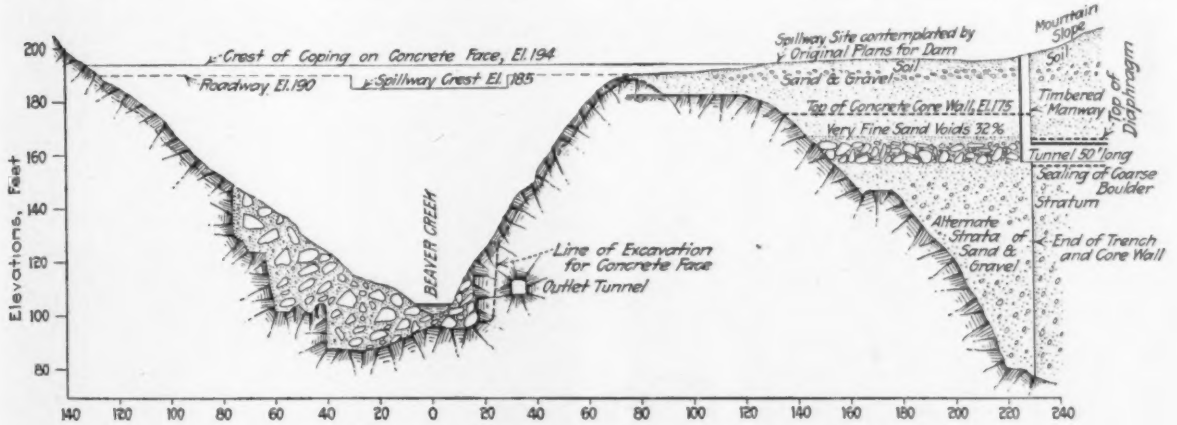


FIG. 3. CROSS-SECTION OF BEAVER PARK DAM SITE, LOOKING UPSTREAM
(At right, the cutoff work done in the pervious strata; at left, abutment)

DESIGN OF DAM

The specifications covering the construction of the dam (Fig. 2) provided for a structure 77 ft. in height above the creek bed. This was later increased to 87 ft. The downstream face of the dam was to have a slope of 1½ to 1, finished to general uniform lines with loose quarry rock. The upstream face has a slope of ½ horizontal to 1 vertical, and is faced with reinforced concrete, which varies from a maximum thickness of 2 ft. at the toe to 1 ft. at the crest. The width of the crest is 16 ft. Since this was the width originally planned and retained when the height of the dam was increased it was necessary to increase the slope on the downstream face for the last 10 ft. of the height from a 1½ to 1 to a ½ to 1 slope.

The downstream half of the dam is composed of rough stone dragged or carried and dropped in place by means of a dragline operating heavy boiler-plate stone boats, skips and rock tongs.

The upstream portion of the dam, with the exception of a wall 5 ft. in thickness forming the water face, is constructed of the same material, derrick placed. This material consists of stone ranging in size from large spalls to 5-ton rock dropped in place with no special attention to securing great mass or density in the structure, and as a result large holes or voids occur throughout the lower three-quarters of the dam. The upstream 5-ft. face is composed of carefully hand-laid dry rubble masonry composed of quarry-faced rock. Unusual care was given to this work and to tying this wall into the loose derrick-placed rock by means of large and long "headers." This hand-laid wall rests upon bedrock both in the bottom of the cañon and on the side walls. Against this 5-ft. wall the concrete face was placed, no rods or ties being used to secure it to the rubble face of the dam, as the interstices in the rubble face were depended upon to give sufficient bond between the concrete and the hand-laid wall.

This concrete face, which is to render the structure water-tight, is reinforced horizontally and vertically with wire fabric of diamond mesh, the main wires being No. 4 gage, spaced about 5 in. apart. No expansion joints were provided, and so far none seem to have been required, for although the concrete face has been exposed to as severe temperature conditions as may be expected, few or no temperature cracks have occurred.

PREPARATION OF FOUNDATION

The specifications provided that in the preparation of the foundation of the dam, only the brush and other vegetable covering on the downstream half of the site should be removed from the surface, but that all brush, dirt and loose material above the center line of the dam should be removed, and the rock formation cleaned, "preferably by means of water used under pressure through a hose." The specifications do not clearly state that this part of the foundation was to be carried down to solid bedrock, but the writer has interpreted the intent to be such. In any event, the excavation for the foundations of the dam above the center line, and for a distance of 25 ft. below the center line, were, or appear to have been, carried down either to bedrock or to a mass of large boulders upon which the first loose rock-fill composing the dam was placed. The specifications also provided that the bottom of the concrete face of the dam should be carried down to bedrock in the form of a toe or cutoff wall. The specifications seem to have contemplated finding bedrock at about 6 ft. below the creek bed, and provided that all work for carrying the cutoff wall to a greater depth should be done by force account or at cost plus 10%.

The additional cost of going to a maximum depth of 21 ft. to get bedrock proved to be very great, although no unusual conditions were encountered. This excavation

was made in a sheeted trench about 3 ft. in width, the material being removed by means of buckets on hand ropes. The seepage water was handled by means of a steam-driven 2-in. vertical centrifugal sinking pump. This work was done during cold weather, which probably contributed somewhat to its excessive cost. During the progress of this work the natural flow of the stream was diverted through the tunnel forming the outlet from the reservoir. This outlet tunnel or conduit consists of a bore 5x6 ft. in cross-section driven through the rock formation under the left abutment of the dam. The upper 60 ft. of tunnel is lined with plain concrete from 6 to 12 in. in thickness.

Construction Details

The concrete in the lower portion of the wall forming the water face and in the gate tower was of 1:2:4 mixture, the aggregate consisting of crushed trachite, while the upper portion of the wall and tower was made of a mixture consisting of practically equal parts of sand and gravel. Up to a point about 20 ft. below the crest, safeguard against accidents. This additional freeboard was provided by carrying the concrete face of the dam up 3 ft. higher than planned, which gave a freeboard of 4 ft. above the roadway. This wall is reinforced with square bars both vertically and horizontally, and is finished with a coping at the top for the sake of appearance. The downstream side of the crest is protected by means of a heavy timber railing of red spruce, the posts of which are set in a reinforced-concrete coping resting on and at 6-ft. intervals extending down into the hand-laid rubble masonry forming the downstream face of the dam. Guard walls of rubble masonry laid in cement mortar, resting on bedrock, are provided at either end of the dam for guiding traffic.

The controlling works consist of two ordinary ribbed cast-iron, bronze trimmed rectangular gates located in the reinforced-concrete tower at the upper face of the dam. Only one of the gates is used for controlling the outflow from the reservoir, the other being placed for emergency purposes. Both gates are operated by rising vertical shafts which are controlled by double-geared stands on top of the tower. Access to the top of the tower is had by means of a reinforced-concrete footbridge extending from the parapet to the tower.

Fig. 4. Looking Downstream at Beaver Park Dam; Near Completion
(Promontory which caused trouble on left)

Ceresit waterproofing compound was added to the water used to gage the mixture. The specifications provided that one part of the Ceresit compound was to be added to an equal amount of water and thoroughly dissolved, after which 11 more parts of water were to be added and this solution used in mixing the concrete for all 24-in. walls, and a somewhat weaker solution used in thinner walls. The results obtained by using this compound seemed so unsatisfactory to the writer that its use was ordered discontinued, and extra cement was added to the concrete at the same cost, which gave much better results.

The original plans provided for a freeboard of but 5 ft. between high-water line in the reservoir and the crest of the dam. While it was not considered essential for the safety of the dam to provide a freeboard of the usual height, yet, on account of the fact that a highway crosses the dam, the writer considered it essential to provide a greater freeboard to protect traffic against the action of spray at high stages of storage and to furnish a greater

The greater part of the material for the construction of the dam was taken from a quarry located on the face of the mountain just above the right-hand end of the dam. During the early stages of the work a quarry was opened on the face of the promontory forming the left abutment. This quarry, being below high-water line, has produced, or at least aggravated, serious leakage conditions. The material lies in broken up and fissured ledges, and was loosened by means of black powder blasts placed in deep holes and "coyote" drifts. The stone was handled from the quarry by means of draglines extending across the cañon over the dam.

Foundation Investigations

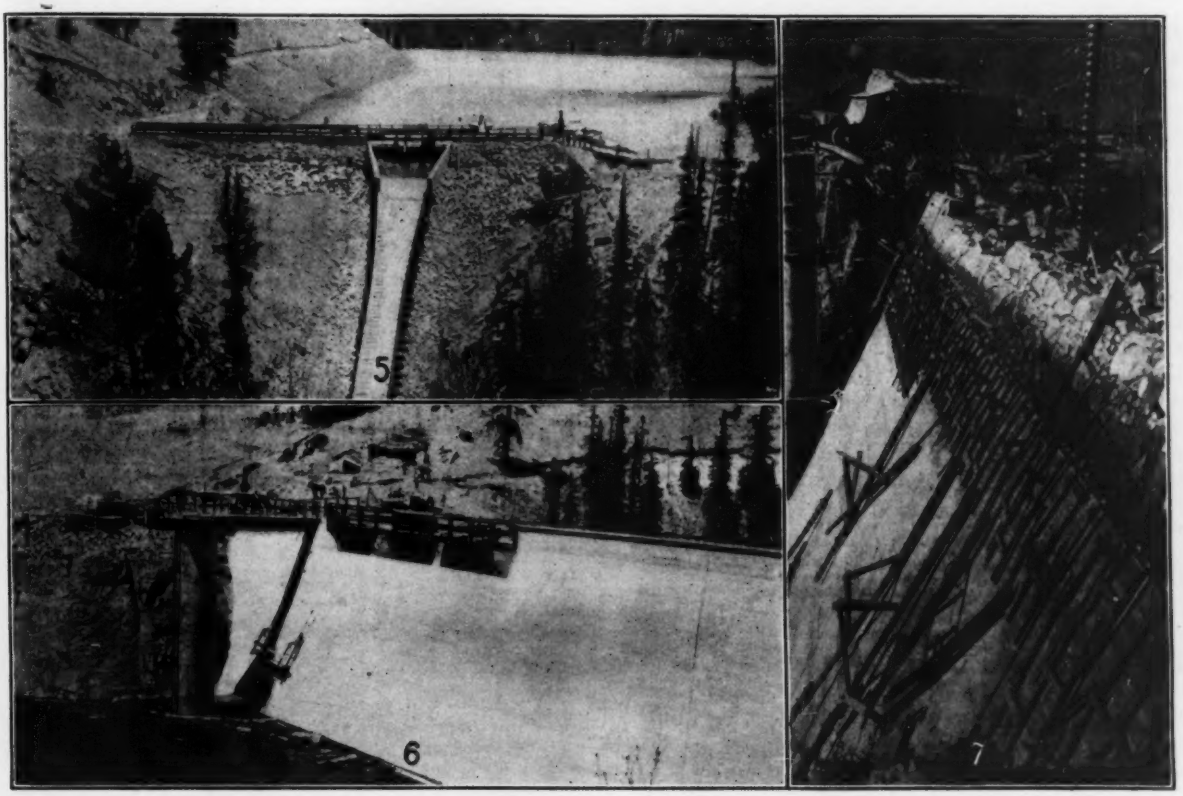
The original plans contemplated a spillway 80 ft. in width excavated in the rock promontory forming the left

abutment of the dam, where a superficial examination would show the conditions to be favorable for such a structure. The rim rock forming the wall of the cañon at the left-hand end of the dam disappears about 100 ft. below the dam in the surface covering of soil and vegetation. At this point (Fig. 1) a short, narrow and very steep side gulch, leading down from the proposed spillway site, enters the main cañon. Immediately below the mouth of this gulch the rim rock of the cañon wall again appears, and hence it was assumed that the rim rock was continuous from the left abutment of the dam to the main wall below the gulch, and that the solid rock formation of the cañon wall would be found immediately below the soil covering the spillway site.

Two days prior to the time the writer was retained as supervising engineer on this work, four test pits had been started and some borings made along the narrow saddle or promontory to actually determine the depth to bed-rock. These test pits and borings showed rock at a depth of about 25 ft. below the surface. Not being satisfied with the assumption that bedrock had been reached, the test pits were ordered sunk to greater depths, with the result that when finally completed it was found that bedrock pitched downward from the surface outcrop near the left abutment at an angle of about 45° away from the dam and toward the main mountain slope. That which was formerly supposed to have been bedrock was nothing but a fine quality of sand with stratifications of coarse gravel, extending from inside the reservoir to the downstream face of the saddle or cañon wall, while the material 25 ft. below the surface located by previous test pits was a stratum of very large boulders, cobblerock and sand from 8 to 10 ft. in thickness (Fig. 3). This saddle is a narrow ridge or backbone about 30 ft. in width on top with a slope of 2½ or 3 to 1 on the reservoir side and 1½ to 1 on the downstream side, and hence would not be impervious to the passage of water from the reservoir with the consequent serious danger from slips when thoroughly saturated. This condition made it necessary to change the location of the spillway and to provide some means for intercepting the percolation of water through the gravel strata extending at various depths through this saddle.

The first difficulty was overcome by placing the spillway over the top of and near the left-hand end of the dam. This structure consists of a funnel-shaped opening 60 ft. in width over the crest of the dam (Fig. 5). The sidewalls are built of rubble masonry laid in cement mortar, while the floor is of reinforced concrete. On account of the expected settlement of the rock-fill forming the dam, it was not thought advisable at that time to build the remainder of the spillway leading down the lower face of the dam of rubble masonry or concrete, and hence this portion of the spillway was built of a good grade of red spruce lumber. The sheathing is of double thickness of 2-in. timber laid with broken joints and painted with tar. It is needless to say that such a structure would not be suitable in the case of an earth dam, but in this instance a very considerable percentage of leakage through the spillway can do no damage to the dam.

A test pit located at the junction of the aforementioned promontory and the main mountain wall was carried down to bedrock, which was reached at a depth of 121 ft. below the surface. At the point where this test pit was



FIGS. 5-7. VIEWS OF BEAVER PARK DAM

(Fig. 5—Looking upstream at finished dam. Saddle or promontory where spillway was first located shown at right. Fig. 6—Near view of upstream completed face. Fig. 7—Concreting the upstream face.)

located the side of the mountain, which is composed of sand and gravel with intermittent boulders, rises rapidly and hence would render further test pits or surface trenching in that direction very expensive. From this test pit a trench was carried backward toward the left-hand end of the dam and excavated to bedrock. This trench was about 3½ ft. in width by 130 ft. in length and ranged from about 3 to 121 ft. in depth. It was necessary to sheet it throughout. At the deepest point reached, bedrock continued at practically the same dip of 45°, with no information being available of where it finally connects with the main wall of the mountain.

Precautions against Seepage

For the purpose of intercepting the seepage from the reservoir through this material overlying bedrock, a concrete diaphragm 2 ft. thick at the base and 1 ft. thick at the top was carried from bedrock upward to a point 10 ft. below high-water line in the reservoir, the full length of the trench. The timber sheeting on the reservoir side of this wall was carefully backfilled with sand and earth and left in place, while the downstream sheeting was removed as the concrete diaphragm was carried up and the remainder of the trench backfilled with puddle. Wooden panels were used for the downstream forms in placing the concrete in the trench. These forms were gradually "pulled" or raised by means of block and tackle as the concrete was poured and the backfilling along the back of the forms proceeded. Before any concrete was placed, a trench about 1 ft. in depth was excavated in bedrock and all seams in the latter cleaned and filled with mortar. Above this concrete diaphragm the trench was filled to the surface with equal parts of clay and sand mixed in a concrete mixer and run into the trench in the form of puddle. The outward end of the concrete diaphragm was built in the shape of a "T" with side buttresses extending 18 in. out from the face of the diaphragm for the purpose of intercepting seepage along the upper face, and also to lend a degree of strength to the wall at that point.

Extending outward from the concrete diaphragm and in a line with the latter, a tunnel 8 ft. in height and 7 ft. in width was driven along the stratum of large boulders and gravel for a distance of 50 ft. into the mountain. The concrete diaphragm was then extended the full length and height of this tunnel, and carried well into the fine material forming the roof and floor and thence into the breast. At the breast of the tunnel it was assumed that the great width of the saddle or promontory would prevent excessive seepage from the reservoir. It would have been advisable to extend this diaphragm until the buried rim rock of the main cañon had been found, and to have provided a drainage conduit back of this diaphragm to carry away the seepage accumulating against the same, but financial conditions prohibited such work at the time. For the purpose of detecting seepage along the diaphragm a 1½-in. pipe equipped with a pump point was placed in the angle formed by the buttress and diaphragm. This pipe extended from bedrock to the surface.

The inner slope of this earth saddle was then paved with hand-laid stone bedded in about 6 in. of gravel to prevent wave action.

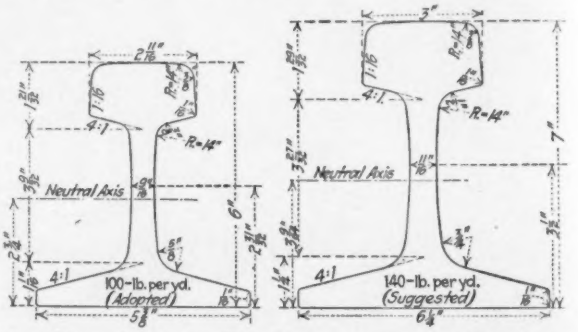
This work was all completed in the spring of 1914, just prior to the approach of the flood stage in Beaver Creek. The gates of the dam were closed and the water allowed to rise against the face of the dam during the two months.

A discussion of the leaks occurring through the mountain walls and the means adopted to overcome the same will appear in a subsequent article.

New Standard Sections for Heavy Rails

One of the most important and useful pieces of work ever accomplished by the American Society of Civil Engineers was the adoption in 1893 of a set of standard sections for railway rails. The sections represented the work of the ablest railway engineers of that day and rapidly superseded the multitude of sections previously in use, representing the divergent ideas of individual engineers.

The principal defect in the Am. Soc. C. E. sections was the unequal distribution of metal in the head and the flange. This caused the rails to assume a curved shape in cooling, and, what was more important, compelled the stopping of the rolling process as soon as the foot of the rail cooled and before the metal in the head had received sufficient working. These were the chief faults that caused the American Railway Association in 1908 to adopt new sections for rails of 60 to 100 lb., in which the metal in the head and foot was more evenly distributed. Unfortunately the Association at that time adopted two sets of standard rail sections instead of one, the two sections representing the divergent views of two factions of the Association's Rail Committee. Of the two sections



New Rail Sections Recommended by the American Railway Engineering Association

(The adopted "standard" sections are 100, 110 and 120 lb. per yd. The tentative sections are 130 and 140 lb. per yd.)

adopted, one type, known as the A sections, was tall and slender while the B sections were short and thick. The assumption was that both sections would be used and a few years' experience would determine which of the two types of sections was the better.

More recently, the American Railway Association requested the American Railway Engineering Association to investigate the experience with these two types of section and recommend a single set of sections for adoption as standard. The use of the A. R. A. sections, especially of the B, or short and thick, type, has been quite limited, and little or no accurate comparison of the service of the two sections has been possible. At the recent annual convention of the American Railway Engineering Association, however, new standard sections were submitted by the Committee on Rails and adopted by the Association. The dimensions of the 100-lb. rail are shown herewith, as are also those of a 140-lb. rail submitted by the committee as a suggestion but not adopted as a standard.

The new sections resemble the slender A type of A. R. A. sections recommended in 1908, rather than the short and chunky B type. It may doubtless be assumed that the latter will be permanently shelved. The committee points out that the strength of the rail as a girder has governed it in its choice of the type of section. It will be noted that all parts of the head, foot and web are heavy enough so that the rail can be thoroughly rolled before any parts begin to reach a blue heat. It will be noted that the new sections are for 100-lb. rails and heavier. For the 90-lb. section the committee recommends that the American Railway Association's A or high section should be the standard, and that for rails of lighter section no change is needed from the present standards.

A Federal Law Protecting Government Employees

By Richard D. Micou*

Prior to 1883 there were no regular rules as to the appointment and removal of Government employees, the matter being left to the discretion of the various Cabinet officers, who in turn were influenced in their action by members of Congress. In 1883, Congress passed the Civil Service law creating the United States Civil Service Commission and specifying the rules under which employees in the "classified" service should be appointed. The classes of employees coming under the provisions of the law have been successively extended by later Congresses and now comprise practically all employees in the government service, except the higher officers, special appointees and employees in confidential and fiduciary positions, such as private secretaries.

Under the original Civil Service law the authority of removal was practically unlimited, the only restriction upon the appointing officer being contained in Section 2 of the act, clause 2, paragraph 5:

That no person in the public service is for that reason under any obligation to contribute to any political fund, or to render any political service, and that he will not be removed or otherwise prejudiced for refusing to do so.

The heads of departments, therefore, had the right to terminate the service of an employee in the "classified service," or out of it, arbitrarily and without assigning any reason, though as a matter of departmental procedure it was usual to submit charges and permit a hearing.

In 1912, however, Congress enacted the following provision as an amendment to the Post Office Appropriation Act (Aug. 24, 1912, 37 Stat. 555):

That no person in the classified civil service of the United States shall be removed except for such cause as will promote the efficiency of said service and for reasons given in writing, and the person whose removal is sought shall have notice of the same and of any charges preferred against him, and be furnished with a copy thereof, and also be allowed a reasonable time for personally answering the same in writing; and affidavits in support thereof; but no examination of witnesses nor any trial or hearing shall be required except in the discretion of the officer making the removal; and copies of charges, notice of hearing, answer, reasons for removal, and of the order of removal shall be made a part of the records of the proper department or office, as shall also the reasons for reduction in rank or compensation; and copies of the same shall be furnished to the person affected upon request, and the Civil Service Commission also shall, upon request, be furnished copies of the same.

As a result of this law, Federal employees under the civil service are now protected against arbitrary action on the part of their superior officers. The decision of such officer, however, is final and the Civil Service Commission is given no authority to review his findings except to see that the removal is in accordance with law and procedure. It follows, therefore, that the superior officer may remove for any causes that unfit the employee for good and efficient service and also for criminal, dishonest or disgraceful conduct, and for political activity.

The provisions of the above act apply only to employees under civil-service rules (except temporary appointments) and exclude all higher officers of the Government and a few employees not subject to such rules.

The act does not, however, apply to suspensions. Apparently at the present time the authority to suspend is unlimited and is frequently exercised for disciplinary reasons or pending investigation and removal, suspension being with or without pay at the discretion of the proper officer. The suspension cannot terminate in dismissal except in accordance with the terms of the above act. The rules of the Commission limit such suspension to 90 days.

While the law quoted is valuable, as stated above, in protecting competent employees from arbitrary dismissal for personal or political reasons, it also operates to make difficult the elimination of incompetent or mediocre men when once installed in the service.

In order to provide for the dismissal of men who become for any reason incompetent, it has been proposed that the present Civil Service Commission should be placed in charge of removals and reductions as well as of appointments.

This has been done in some states, notably Illinois, with marked success and has resulted in increased efficiency in the state civil service, the number of removals for inefficiency being doubled as soon as the appointing officer was relieved of the duty of hearing cases.

In the Federal Service the tendency would be to remove the Civil Service employees still further from political control. Under the present system, for example, a bureau chief reports to the Secretary that a certain clerk is inefficient. The Secretary starts an investigation and the clerk, hearing of it, goes to his Congressman, who immediately exerts influence on the Secretary to retain the clerk, which he usually does. The result is that inefficient clerks are retained through political influence to which a trial board would not be amenable.

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Credit to Civilian Engineers as well as to the Army engineers for the building of the Panama Canal was referred to in an editorial in our issue of Mar. 11. The "Panama Star and Herald," on Mar. 10, printed the following remarks by Gen. Goethals at a meeting of the Society of the Chagres: "After eight years of responsibility I feel that I am entitled to some rest, so I have asked to be relieved and hope that I may be some time during the coming year in order that a younger man whose energy is unimpaired may take up the work and carry it forward. One subject I desire to touch upon is the promotion given to the officers of the Army, Navy and Marine Hospital Corps. Mr. Taft in his message of 1912 recommended that I be promoted to the position of Major-General in the Army. Two friends of mine in Congress, one in the Senate and one in the House, immediately introduced bills for that purpose. I wrote and told them service men should have no promotion unless promotions could be given to all alike. That attitude I have consistently maintained. It is a source of regret to me that the bill ever passed. Civilians built the canal, not the Army. When the Army people leave here they are retired and looked after by the Government, whereas the civilian must get out and hustle to make a living for the rest of his days, and I have always opposed it. The completion of the canal is due to the brain and brawn of the men who are gathered here."

Methods Pursued in Tearing Down a Large Building

By Joseph Rosenzweig*

In large cities the construction of a new building of any size generally means the destruction of some old building on the site, and in congested districts this old building may be of such size that its razing in itself is quite an engineering problem. The following notes on the razing of a large building in New York City may be of some service to engineers contemplating similar work.

The contract for the demolition of the old Mutual Life Insurance Co.'s building, erected in 1863, at the southeast corner of Broadway and Liberty St., was awarded to the firm with which the writer was connected in 1911. Some of the more important terms of the contract were: The completion of the work in 40 working days, a forfeiture of $100 for every day of delay after contract time; a requisition clause permitting the owner to remove whatever he desired from the condemned building at his own expense; the complete destruction of the building down to and including the lowest tier of beams; the removal of all débris; leaving the cellar clean.

Preliminary Operations

The first step after getting the contract was to obtain the necessary permits for executing the work. They were as follows: (1) Permit for the demolition of buildings, from the Bureau of Buildings; (2) permit to place building materials on street, from the Bureau of Incumbrances; (3) permit for the use of water, from the Dept. of Water Supply, Gas and Electricity; (4) permit for erecting temporary roof and shed, from the Bureau of Highways; and (5) special requisitions as found necessary, for permission to work on Sundays or holidays, from the Police Department.

It would be commercial suicide to undertake the wrecking of a structure such as we are considering without accident and liability insurance to cover it, for the accidents are many and inevitable. The policies covered all employees and the general public, for all accidents whether trivial or serious, but in every instance it was to be proved that the injured person was in no manner guilty of contributary negligence before damages could be recovered.

Bonds for the sum of 25% of contract price were given by the wreckers for faithful service, and were cancelled on completion of work.

In the house-wrecking business there are to be found the following divisions of labor: (1) Barmen, who use either a crowbar or sledge hammer and wedge; (2) laborers, who clean bricks or shovel the débris; (3) rockmen, who break stones or do other extremely hard work; (4) carpenters, who build the scaffolds, the wooden centers, the sheds and shanties; (5) ironworkers, who manipulate the derricks and handle all the iron in the building. Under management are included employers, foremen, timekeepers, bookkeepers, all of whom assist in managing the wrecking operations and directing the workers. Usually all large house-wrecking concerns have a small standing army of workmen procurable at a moment's notice, and if a larger force be suddenly needed, the foreman has but to notify this little army and the next day sees hundreds more on the job; this is the common practice and more effective than advertising, since almost none employed in this class of work can read or write.

In order to know how best to proceed with the work of demolishing this massive structure, it was necessary to obtain some information regarding its construction, which was accomplished in the following manner:

To ascertain the character of the front walls, a hole was cut through each of them near a window frame, which revealed the following detail of construction: Behind ¾-in. plaster was a 4-in. row of terra-cotta hollow tile, followed by a 4-in. air space; then came a brick wall 2 ft. thick to which was fastened the ashlar masonry, varying in thickness from 6 to 12 in., making a total average thickness of 42 in.

The partition walls were similarly penetrated near a door frame or window and were found to be composed of bricks laid in cement-mortar, 20 in. thick, with furring and plaster adding an inch to each side, making a total thickness of 22 in.

The floors were next investigated and revealed a peculiar form of construction. Underneath the wooden floor of spruce ⅞ in. thick came wooden sleepers ¾ in. thick, spaced 2 ft. c. to c.; these rested on sandstone slabs 1 in. thick, 1 ft. wide and 2 ft. long, underneath which came 2 in. of concrete, overlying brick arches resting on the bottom flanges of 9-in. I-beams; then the ceiling of the lower floor, composed of dovetailed corrugated tin, bolted to the same flanges with ¼-in. bolts, added 1⅛ in. to the depth of floor, making a total of 14¾ in. From the foregoing figures and the floor areas, the cubical contents were computed to be about 10,000 cu.yd. or about 2500 truck loads.

Removing Adjacent Buildings

In addition to the old Mutual Life Insurance Co. Building, a brownstone building on Liberty St. and a cast-iron-front, brick-interior dwelling house on Broadway were included in the contract, and the former was attacked first. When it was half demolished, another force of men was distributed in the old Mutual Life Building to prepare it for final demolition by stripping the interior of everything of value, and when this was accomplished another force was distributed in the cast-iron-front building for a similar purpose.

At the same time small forces of plumbers and electricians were engaged in running pipes and wires through the buildings for water-supply and lighting purposes. Sites for chutes were selected, and timber for scaffolds and bridges was prepared by a force of experienced rough-hand carpenters. All the timber used came from the two smaller buildings, except the girders and struts used to support the heavy bridge over the sidewalk.

As it was ripped off with bars, the tin roof covering of the brownstone building on Liberty St. was cut up into sections 3 ft. wide and 6 ft. long; these were rolled up and lowered to the street, where they were piled up to await removal.

The left wall was a two-party wall, and hence great care had to be exercised in removing the floor-beams so as not to injure it. The holes were afterward refilled with brick and mortar and then covered with tar paper. The regular procedure after the roof was off was to tear the walls down to the next floor level, then rip up the flooring, remove the beams, and then tear the walls down again, etc.

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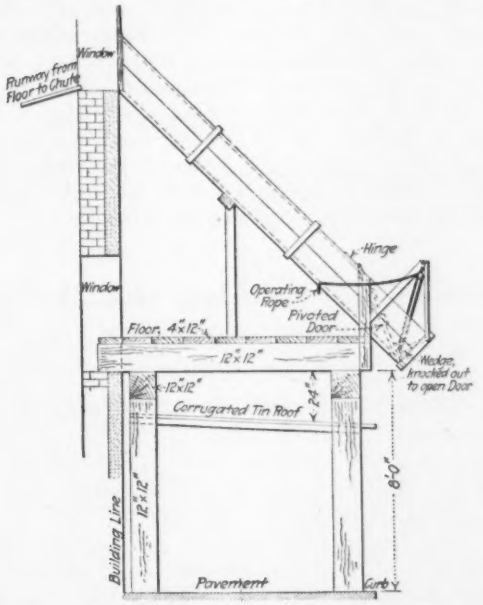
While the stripping of valuable material was going on, before demolishing operations began, the window openings were boarded up tightly to keep débris and dust off the street. A second provision for this purpose, which incidentally is required by law, was to provide plenty of water to sprinkle about the falling and fallen débris, and this is necessary for the ease of the laborers and comfort of the public.

Character of Main Building

The old Mutual Life Building was seven stories high, exclusive of tower and superimposed clocks. It consisted of the floors described above resting on cast-iron columns, with stone- and brick-bearing walls. It was massive and heavily built with three safe-deposit vaults in its interior. The sixth and seventh floors were built in mansard fashion with bay windows. The roof was of tin, covered with tar and gravel, and had a copper cornice.



Details of Débris Chute and Sidewalk Bridge for Razing Building

From 3 ft. below the street level to the cornice on the sixth tier, the ashlar was of Georgia marble. The front walls of the sixth and seventh stories were of brick backing covered with slate shingles fastened to corrugated tin, which in turn was bolted to 1½x1½-in. iron angles and these again were bolted to 6-in. I-beams, forming a part of the structural frame.

Bridges, Chutes and Scaffolds

As the law requires that traffic be not interfered with by storing material around the premises, and the public be not endangered by falling material, a bridge was built completely around the three buildings, extending from the house line to the curb. This bridge was of heavy yellow-pine timber (shown in the accompanying figure) spiked with nails throughout. It was 8 ft. high. Two feet below its top was constructed a hanging ceiling of corrugated sheet iron resting on scantlings nailed to the uprights. A board fence inclosed the entire bridge above and it was electrically illuminated below.

Immediately after the bridge was built, the construction of the scaffolding was begun on the fifth floor to permit the removal of the towers on the sixth and seventh floors. This scaffolding consisted of wooden planks laid across I-beams projecting from all the windows and counterbalanced by means of collapsible clamps attached to the floor-beams within. As each floor was removed, the scaffold was lowered, half of it at a time, to the floor beneath, so as not to interrupt the work.

Upon the completion of the first scaffold the carpenters erected four chutes from as many windows on the second floor, one facing Liberty St., one projecting into the lot made clear by the demolition of the smaller house on Liberty St. and two projecting into Broadway. All extended over the bridge (at an angle of 45° with the wall) for 3 ft., to enable the trucks to back in and receive their loads of brick, dirt or other material, without extra shoveling. By means of a trap door at the end of each chute, material could be retained until a truck was properly backed in.

While this carpentry work was going on a large force of barmen and rockmen was breaking through the brick arch floors with 24-lb. sledges. The scheme followed was to demolish the third tier arches, allowing the débris to fall to the second floor; the higher floors were demolished in turn and last of all the second floor, which had remained intact to serve as a platform on which to handle the débris as it accumulated.

As the material was falling all day no shoveling could then be done, so a night shift was organized to shovel the accumulated débris into specially constructed push carts, in which it was carried to the chutes by the same men who filled them, and dropped through into the waiting trucks below.

Derrick Work

In house wrecking the derrick has found a new field of usefulness. For pulling down walls and piers, removing beams from their anchorage, and lifting columns from their fastenings, it possesses great advantages over every other means.

The ordinary method of tearing down walls is either to drive wedges into them and break off pieces varying in volume from one-half to one cubic yard or more, or else to loosen each brick with a bar. The derrick enabled us to swing out a 600-lb. cast-iron ball, suspended from the boom, as far as possible and allow it to strike piers or walls, bringing them down in a fraction of the time otherwise consumed. The capacity of the derrick used in this case was 10 tons, and as a very strong foundation for it was needed on account of the concentrated load brought on the floors; this was furnished by underpinning each floor on which the derrick was placed, and using an extra long wale on which to rest its base. The steam hoisting engine used was located inside the building on the basement floor. The guys which held the mast were tied to wall-bearing floor-beams, one floor below that upon which the derrick rested. In shifting the derrick from floor to floor, the mast is first used to lower the boom, then the boom is used to lower the mast. It required between three and four hours to manipulate and make a shift.

Handling Iron and Stone

In these buildings there were approximately 500 tons of scrap iron, 100 tons of girders, 200 tons of columns and 150 tons of beams, and only the use of the derrick

enabled the wreckers to handle them economically. The scrap was collected in two ordinary skips of one and two tons' capacity, lowered by the derrick, and dumped into wagons. The girders, columns, and beams were lowered onto trucks—the beams in piles of ten and twenty, and the columns and girders singly—all without rehandling, and hauled away. There were three courses of stone cornice, scores of voussoir and numerous cast-iron lintels to be removed and lowered. Each of the courses of cornice had more weight overhanging the street than in the brick wall behind the ashlar, and to loosen it without letting it fall was accomplished by notching the joints and securing each piece with 1-in. manila rope, the free end of which was tied around a floor-beam; the brick behind the piece of cornice thus secured was then torn away and each stone pulled back into the building.

As the demolition of the walls progressed it became necessary to support the window arches and for this purpose wooden centers were used, which were either supported on vertical scantlings or wedged in between window piers, as conditions required. Nearly all the skylight windows had cast-iron lintels of various sizes, and as they were reached the derrick picked them up, until a pile of about ten or twelve was made, when they were lowered to a junk wagon in the street.

Cellar Work

In demolishing any building, the work from the roof down to the second or first floor is not as troublesome as from there on, because, by a cumulative process that is entirely uncontrollable, various material piles up in the cellar, basement and first floor, to such an extent that the quantity of débris is almost doubled on these levels. Furthermore, as in the building under consideration, heavier columns, girders and beams were encountered, which materially increased the work and delayed its time of completion.

When the chutes were broken up, the derrick and a half-dozen buckets took their place, and when it became necessary to clean up the cellar, the buckets were lowered into it, filled with débris, hoisted out and dumped directly into wagons of three and four cubic yards' capacity.

Demolition of Vaults

On the ground floor an immense safe-deposit vault had to be demolished, and two others of smaller size on the first floor. From their construction, it was apparently intended that they should stand forever. The largest was 30x30 ft. by 10 ft. high; its walls were of granite resting on solid ground, and its roof was made of chilled iron between steel plates riveted throughout, and built in 3x6-ft. sections, resting on I-beams. On top of this was the regular brick arch floor. One barred window and two iron doors opened into it. The arch floor was first destroyed; then by means of a backing-out punch the plates were disconnected from the I-beams, the blocks of granite were separated from each other by steel wedges, and, piece by piece, were lifted onto trucks by the derrick. The smaller vaults were built of brick, laid in cement mortar, with granite floors.

Condition of Material

From an engineering viewpoint it may be more than interesting to consider the condition of the structural materials after a service of nearly 50 years. All material was personally inspected by the writer as it was removed and the following facts noted: The cast-iron bay windows showed a thick coat of rust on their surfaces; the girders exhibited slight traces of rust where the paint had scaled off. The oxidation was more evident on the beams than on the girders, especially at the ends. The first-tier beams had been painted with a coat of red lead when erected and were in a remarkable state of preservation. The columns were found to be in good condition on the outside, having been protected by several coats of paint, but badly rusted on the inside, and on being broken numerous flaws were discovered in the metal. One column showed a shell thickness of ¼ in. over a space about 1 ft. long and 4 in. wide. The lumber was in most cases in excellent condition but very dry, and in several instances the ends were badly decayed. The marble ashlar, when broken, showed weather effects to a depth of $\frac{1}{32}$ in. and extraordinary brittleness. The cement mortar used in some of the walls proved harder than the bricks and manifested a stronger adhesion between itself and the brick than existed between the molecules of the brick. These bricks could not be cleaned, but those laid in lime mortar offered no resistance to the process of cleaning.

Curves for Solving the Hydrostatic Catenary

By H. M. Gibb*

If a flexible sheet, with no resistance to bending, is hung between two level supports, and the resulting basin filled with water up to the level of the supports, the sheet will hang in the curve of a *hydrostatic catenary.*

The characteristics of this curve are:

1. The tension T in the sheet is a constant because the pressure of the water is normal at all points to the surface of the curve.

2. There is neither shear nor bending moment induced at any point.

3. The radius of curvature is inversely proportional to the depth; for, resolving forces normal to the curve at any depth y:

$$T\,d\phi - w\,y\,ds = 0$$

or

$$wy\frac{ds}{d\phi} = T = constant$$

Therefore $\frac{ds}{d\phi}$, which equals the radius of curvature, must be inversely proportional to y.

The solution of the equation involves the use of elliptic functions, but from this last property an easy graphical method of drawing the curve can be derived (see G. P. Manning in *Engineering,* July 31, 1914). Its construction is shown in Fig. 1, and graphs founded on this construction are given in Figs. 2, 3 and 4.

From Fig. 1 it will be noticed that to get the curve with certain particular proportions it is necessary to choose the initial radius r_1 properly in order that the construction may finish by the arc of r_{10} crossing line 10 at the proper point.

In Figs. 2 and 3 the various properties of the curve are given for values of the ratio, b/d from 0 to 2.0.

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Curve 1, Fig. 2, gives the initial radius r_1 in terms of the maximum depth d, when n equals 10; $r_1 = m_1 d$.

Curve 2, Fig. 2, gives the half area inclosed by the curve in terms of d^2; $a = m_2 d^2$.

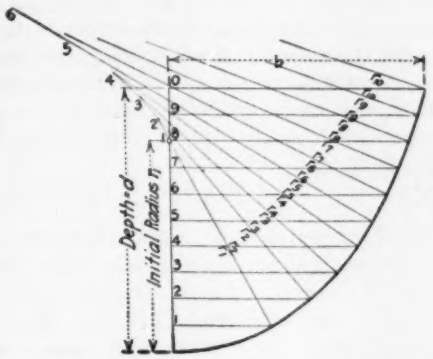


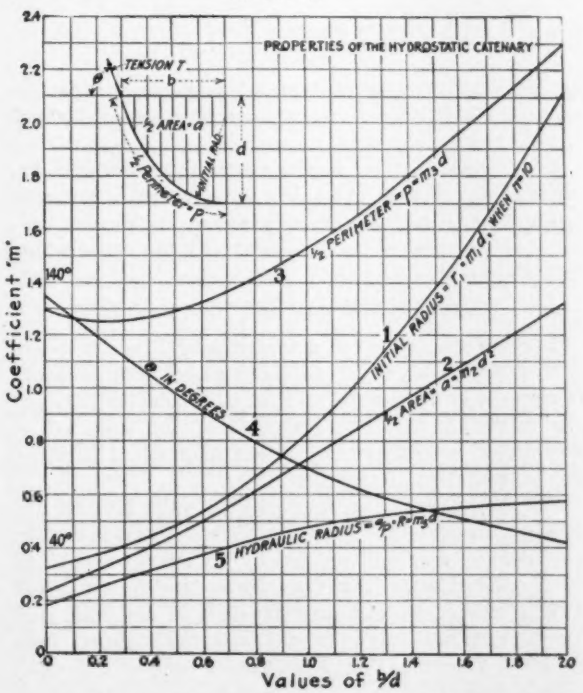
FIG. 1. THE HYDROSTATIC CATENARY

(Radii vary inversely as depth. For 10 divisions of depth d, get initial radius r_1 from Fig. 2. The other radii are found from it, thus:

$$r_2 = \frac{10}{9} r_1$$

$$r_3 = \frac{10}{8} r_1$$

$$r_{10} = 10\, r_1)$$



Curve 3, Fig. 2, gives the half perimeter of the curve in terms of d; $p = m_3 d$. For a given depth, the perimeter is a minimum when $b/d = 0.22$.

Curve 4, Figs. 2 and 3, gives the end slope θ in degrees; for $b/d = 0$, $\theta = 135°$; for $b/d = 0.63$, $\theta = 90°$.

Curve 5, Fig. 2, gives the hydraulic radius in terms of d; $R = a/p = \dfrac{m_2 d^2}{m_3 d} = m_5 d$.

Curve 6, Fig. 3, gives the hydraulic radius in terms of the half area; $R = m_5 d = m_5 \sqrt{\dfrac{a}{m_2}} = m_6 \sqrt{a}$. For a given area the hydraulic radius is a maximum when $b/d = 1.0$, but it decreases only about 10% when $b/d = 2.0$.

Curve 7, Fig. 3, gives the tension T in terms of the area; $T = aw \operatorname{cosec} \theta = m_7 aw$, where w = unit weight of fluid. This curve represents *cosec* θ plotted on b/d.

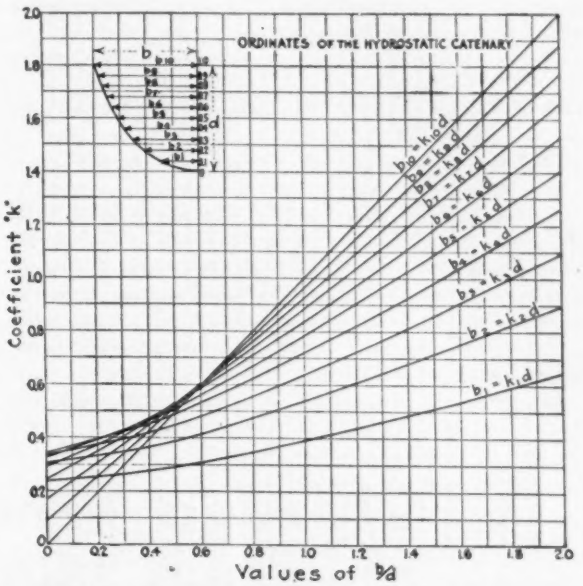
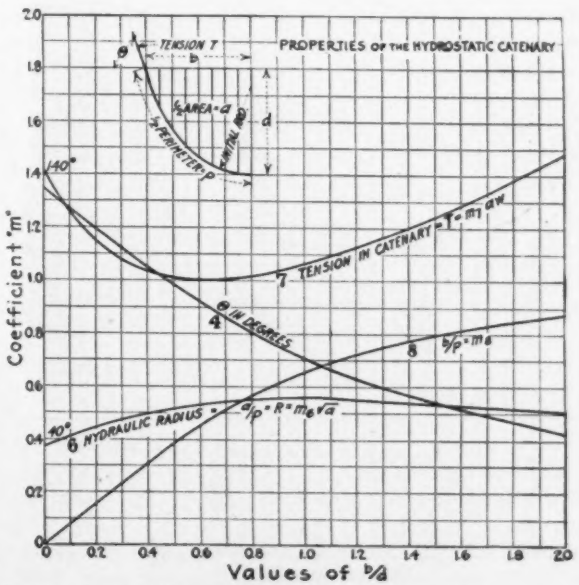


FIG. 4. COÖRDINATES OF HYDROSTATIC CATENARY



FIGS. 2 AND 3. CURVES OF INITIAL RADIUS, PERIMETER, AREA, ETC., OF HYDROSTATIC CATENARY

For a given area the tension T is a minimum when $\theta = 90°$ and $b/d = 0.63$.

Curve 8, Fig. 3, gives the value of the ratio b/p for values of b/d. By this relation, it is possible to solve the case where b and p are given and d has to be found.

USE OF THE CURVES

Given total area = 90 sq.ft., half area = 45 sq.ft.; $b/d = 0.98$. From the curves, $d = \sqrt{\dfrac{a}{m_2}} = \sqrt{\dfrac{45}{0.715}} = \sqrt{63} = 7.94$ ft.

If $n = 10$, the initial radius $r_1 = m_1 d = 0.80 \times 7.94 = 6.35$ ft.; $r_2 = \frac{1}{9}^0 \times 6.35 = 7.05$ ft., $r_{10} = 10 \times 6.35 = 63.5$ ft. The half perimeter $= m_3 d = 1.51 \times 7.94 = 12.0$ ft., and $\theta = 71.5°$. The hydraulic radius $= m_5 d = 0.475 \times 7.94 = 3.76$ ft., or $= m_6 \sqrt{a} = 0.56 \sqrt{45} = 3.76$ ft. The tension $T = m_7 aw = 1.06 \times 45 \times 62.5 = 2980$ lb. per lin.ft.

Again, given $b = 12$ ft. and $p = 18$ ft., find d: From curve 8, with $\frac{b}{p} = \frac{12}{18} = 0.66$, read $\frac{b}{d} = 1.02$; then as $b = 12$ ft., $d = \frac{12}{1.02} = 11.62$ ft.

Fig. 4 gives the ordinates of the curve in terms of d, for values of b/d, and enables the construction of the curve to be made quicker than by construction in Fig. 1. When b/d is more than 0.63, the maximum ordinate is the top one, b_{10}; for values less than this, the maximum ordinate does not occur at the top, but at some intermediate point.

Comparing the hydrostatic catenary with the parabola and the arc of a circle, it will be found that for values of b/d about 1.0 and greater, it lies between the parabola and the arc of a circle, but when b/d is about 2.0 it lies outside the parabola in the upper part and inside it in the lower part.

The following is a comparison of the areas inclosed by the hydrostatic catenary, the circular segment, and the parabola, expressed in percentages of the inclosing rectangle, for varying values of b/d.

Ratio b/d	1.0	1.2	1.4	1.6	1.8	2.0
Inclosing rectangle	1.00	1.00	1.00	1.00	1.00	1.00
Area of hydrostatic catenary	0.73	0.708	0.693	0.682	0.672	0.667
Area of segment	0.785	0.75	0.731	0.716	0.706	0.698
Area of parabola	0.667	0.667	0.667	0.667	0.667	0.667

The application of this curve occurs in structures holding fluids, such as metal flumes, and large uncovered water pipes under a very low head. For spherical bottoms of water tanks and spheroidal gas-holder tanks (see ENGINEERING NEWS, Aug. 6, 1914), the above curves will give a close first approximation to the proper curve, and the correct curve can then be found from this by means of the funicular polygon.

Chicago Track Elevation; Rock Island Lines

SYNOPSIS—This work represents an expenditure of over $10,000,000. Its special problem was the elimination of a railway grade crossing in addition to the street crossings, necessitating an unusually high elevation (for tracks on two levels) and a five-span skew bridge over the lower tracks. Among the interesting features of the construction may be noted the following: (1) The high-level crossing; (2) the concrete-cased steel bridges; (3) the use of bin cars for the concreting trains and traveling forms for the retaining-walls; (4) the carrying of drainage from the fill to chambers formed in the retaining-walls and having sewer connections; and (5) the subway bridges, with steel columns, steel and concrete cross-girders and concrete slab spans.

One of the heaviest pieces of track-elevation work at Chicago is the extension of the track elevation of the Rock Island Lines from 72nd to 90th St., about 3 miles.

The Rock Island Lines were already elevated as far south as 76th St. and descended to the street level at about 78th St., but the high elevation required for a double-track crossing at 79th St., over the Chicago & Western Indiana tracks, made it necessary to begin the new work at 72nd St., increasing the present elevation on a grade of 0.5% to the summit, beyond which a similar grade descends to the normal level of the new elevated tracks. This new work will extend to 90th St., but the portion under construction during the past season only went to 84th St., as it was desired to concentrate the work on the complicated crossing at 79th St.

The work involves a total expenditure of about $10,000,000, and requires the placing of about 290,000 cu.yd. of concrete in walls, abutments and slab-deck bridges, besides about 15,000 concrete piles, aggregating over 300,000 lin.ft.

The special features in the execution of the work include: (1) Careful planning in advance; (2) rapid progress; (3) noninterference with a heavy regular traffic; (4) low cost of construction; (5) the organization and cost-keeping system.

Retaining Walls and Abutments

Between the bridges at street crossings, the tracks are carried on a solid fill of sand and gravel between heavy concrete retaining walls. This is typical of Chicago track-elevation work, but in this case the walls are of exceptional height, reaching a maximum of 42 ft. above the bottom of the foundation.

The foundations for the retaining walls and for the bridge abutments at subways are concrete piles, cast in place in the clay soil and having an average length of 22 ft. The heads of the piles are embedded in a heavy footing course, which forms the base of the wall, and has in it vertical lengths of old rails which project from the surface and thus form a mechanical bond with the wall proper. Load tests were made on the piles, and the actual load carried is 20 to 25 tons per pile.

In the design and construction of the retaining walls, a special effort was made to avoid the unsightly cracks which have developed in some other track-elevation work, and also to keep down cross-section area. The aggregate amount of concrete on the work is very great, and the wall design aims to secure the necessary strength with a minimum of material.

The walls vary in height from 20 to 36 ft., and a typical section is shown at the left in Fig. 1. The face is practically on the right-of-way line (with the toe of the footing projecting over the line). The face is ver-

tical to the coping, while the back is stepped. At some points, however, the footing must be kept within the right-of-way line, and here the body of the wall is set back to allow of giving an outside toe. But the full roadbed width is retained nevertheless by forming a cantilever projection at the top of the wall, supported at 10-ft. intervals by triangular brackets. This form of wall is shown in the middle section in Fig. 1.

The walls are built in sections 35 ft. long, which is considered the maximum length that can be used without fear

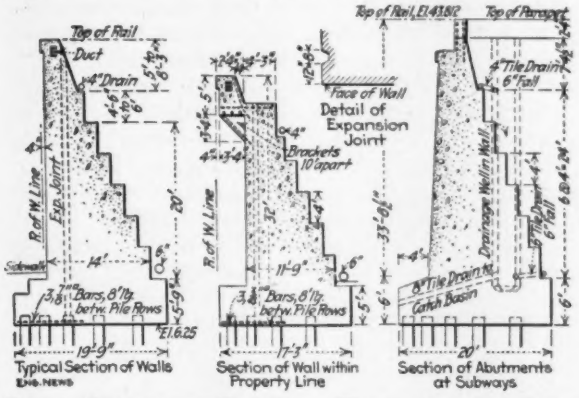


FIG. 1. SECTIONS OF RETAINING WALLS AND ABUTMENTS ON CHICAGO TRACK-ELEVATION WORK; ROCK ISLAND LINES

of cracks due to temperature movement. The sections have vertical keys at the expansion joints. In the top of each wall is a six-duct conduit for electric wires, cables and telegraph lines. Manhole chambers on the conduit are placed about 400 ft. apart, and are 6x3 ft., 4 ft. deep, with a reinforced-concrete slab cover having a 28-in. iron manhole.

An unusual feature is the provision of drainage wells in the ends of the retaining walls, adjacent to the abutments at the subway bridges. These are 3x3 ft., and extend to the bottom of the wall, as shown in the drawing of the abutment cross-section in Fig. 1. There are no weep holes through the retaining walls, but along the backs of the walls are laid inclined drains of 6-in. porous tile, on a grade of 0.5%, extending from subgrade level to 6-in. pipes which are embedded in the rear part of the walls and discharge into the drainage wells. Each well has an 8-in. connection to the catchbasin of a city sewer, as shown.

The waterproofing consists of a tar-pitch composition applied to the back of the wall, a strip of burlap and felt being placed over each expansion joint and well mopped with the composition. An ornamental concrete railing 4 ft. high will be placed upon the parapet over the subway.

At some of the cross streets the tracks were already elevated, but had to be raised still higher, due to the change in grade, as noted above. It was not considered desirable to increase the load on the old abutments, but instead of destroying them it was decided to build a heavy cross wall (pierced with rectangular openings) behind the abutment and supporting the new fill. The space between the two walls, about 17 ft., is covered with a deck of concrete slabs. In this way the load is transferred largely to the new wall, which extends between the retaining walls and has the usual pile foundation. In raising the height of the old abutment, a wall will be built upon it, the lower part of the wall sufficing to retain the toe of the fill and the slab behind it.

The fill is of sand and gravel, dumped from cars. Before the final surfacing to subgrade, the fill will be thoroughly soaked with water. This is done to reduce the amount of settlement after the work is completed and in service, as surfacing track under heavy traffic is difficult and costly.

SUBWAY BRIDGES

With the exception of the 79th St. crossing, noted below, the subways have slab deck spans of reinforced concrete. They are of the four-span type, with bents on each curb-line and at the middle of the street, as shown in Fig. 2. The two street spans are 20 to 23½ ft., with a clear headway of 13½ ft., while the sidewalk spans are 10 ft. The foundations are generally concrete footings on concrete piles. In a few cases, where soft ground was encountered, 44-in. to 60-in. open wells or caissons were sunk to rock (at depths of 40 to 50 ft.) and filled with concrete to form cylindrical foundation piers. In one case there were four piers, capped with a double row of steel box girders carrying the eight columns of the bent. In all other cases there was a pier under each column.

The columns are of steel, composed of an I-beam and two coverplates, with an angle at each corner and batten plates riveted across the angles. These are shown in Fig. 3. Where the height exceeds 25 ft., horizontal box-lattice struts extend between the columns (Fig. 2). All these steel members are wrapped with wire netting, and are filled and heavily cased with concrete. The concrete for this purpose is a 1:2:4 mix, using ¾-in.

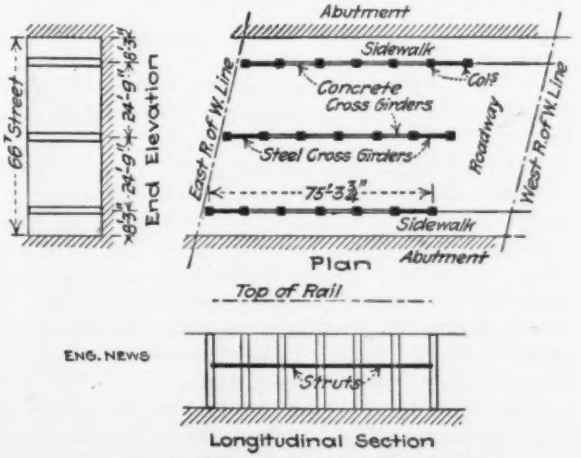


FIG. 2. TYPE OF SUBWAY BRIDGES

stone. Upon the columns of each bent are placed the cross girders, which are shown in Fig. 3. Those beneath the two outer tracks are steel box girders, while the others are of reinforced concrete, but the former are encased in concrete so that the difference is not apparent. The cross girders of the center bent have the under side curved or arched between the columns, but the shallower girders on the sidewalk bents are flat on top and bottom.

The deck slabs rest upon the abutments and the cross girders, and their construction is shown in Fig. 4. The largest slabs are 25x6 ft. and 44 in. thick, weighing about 46 tons each. The slabs of the shorter spans being thinner, are of L section, the ledge on the under side

raising the slab so that its top is level with the top of the roadway spans. All the slabs for the sides of the deck are also of L-section, having parapet walls formed upon them to retain the ballast. The slabs are laid with ¼-in. joints, which are packed with asphaltic filler. Owing to the bridges being on a skew, the parapet slabs are of irregular shape in plan, and each has to be set in its proper position, as shown on the plans for each bridge.

The slabs are cast at a yard near 93rd St., a special organization being maintained for this work. Platforms are arranged on each side of a double-track spur, one track for the concrete mixer and the other for material cars. On these platforms are erected the slab forms, which are mainly of wood, but steel has been used for some parts and will be used more extensively. The concrete is a 1:2:4 mix. The slab gang consists of a foreman and 40 men, and turns out an average of 20 slabs per week.

The steel reinforcing bars (of deformed section) are purchased in commercial lengths, and cut and bent as required. For each cross-girder the reinforcement (including hook for handling) is assembled and wired together at the yard and set in the form as a unit. A concrete mixer with 1-yd. drum is mounted on a car and discharges the concrete into a chute leading to the form, where it is well stirred with paddles to insure thorough filling around the steel bars.

The viaduct crossing over 79th St. and the Chicago & Western Indiana R.R. (where the two roads intersect at an angle of about 17°) consists of five through-girder spans of about 90 ft., on concrete piers and abutments. The girders and floor-beams are incased in concrete, and the floor consists of a concrete slab embracing the top flanges of the floor-beams, while the lower flanges are embedded in a thinner slab, which serves to exclude the steam and gas from the locomotives. This structure will be described in a separate article.

Construction Methods and Plant

Concrete—The concrete is made of gravel which is washed, screened and graded into three sizes: torpedo sand, 1-in. stone and 2-in. stone. This material is obtained from a gravel pit and washing plant near Joliet, Ill. The proportions are 1:3:5 for the retaining walls and abutments, and 1:2:4 for deck slabs and all reinforced concrete.

The material for concrete is brought in by special trains, and in general the cars are dumped from high pile trestles which serve later for temporary tracks while traffic is being diverted. The cars are steel gondolas, with drop doors along each side, being regular freight cars assigned to this work. The temporary approaches to these trestles are of 3% grade. To a considerable extent the material for concrete is dumped directly into bin cars forming part of the concreting trains, the cars standing on a track parallel with the trestle, and the side of the trestle having an apron of proper slope, as shown in Fig. 5. The cement is delivered in sacks, but it is expected to make some trial of cement delivered in bulk.

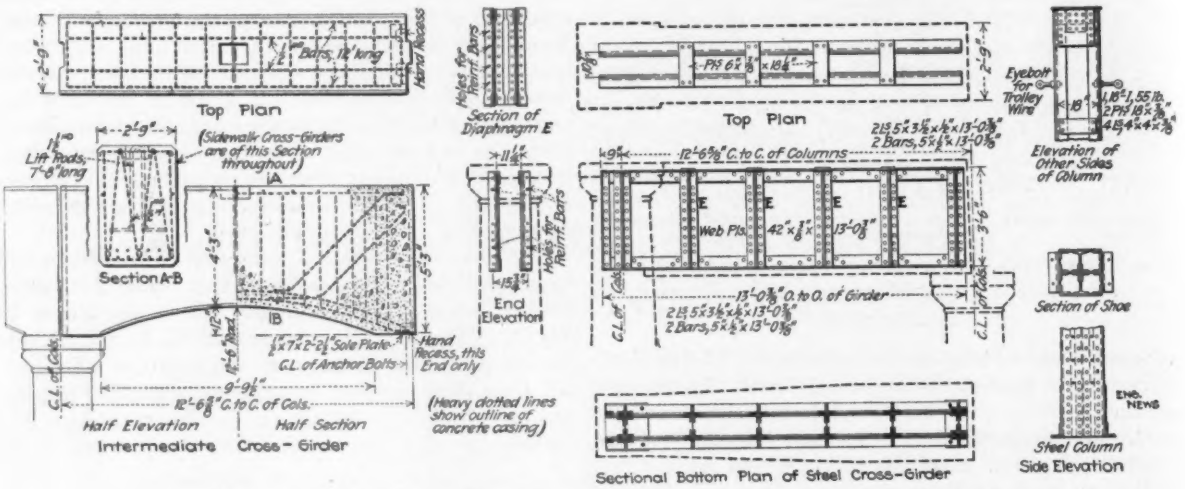


Fig. 3. Columns and Cross Girders of Subway Bridges; Rock Island Lines

Each concreting train, Fig. 5, consists of three steel flat-cars. The head car carries a 1-yd. mixer, feed hoppers, measuring tank, and a bucket-elevator tower 60 ft. high with a hopper and jointed swinging chute to deliver the concrete into the forms. The trains building the foundation course around the heads of the piles had no towers, the mixers discharging either into buckets or directly into chutes. Behind the mixer car are two bin cars, each with capacity for 80 yd. of gravel and 80 yd. of sand. Below each main bin is a measuring hopper, beneath which runs a conveyor belt. The operator first opens the bin gate and fills the measuring hopper, and then opens the gate of the hopper, allowing the charge to fall upon the belt, which delivers it to the feed hopper above the mixer, carrying about 1 yd. of concrete material and 4½ bags of cement. The cement also is delivered to the feed hopper by a conveyor, and Fig. 5 shows the men dumping cement from sacks into a hopper over the conveyor.

The walls are built in traveling forms which straddle the site of the wall and are carried by wheels on either side. Both wood and steel forms of this type are used, each being long enough for a 35-ft. section and having grooved wheels riding upon two lines of rails. These are shown in Figs. 6 and 7.* The abutments are built in fixed forms of the usual type. Plank sheeting is used in both cases, and the two lines of sheeting are held together by tie-rods instead of wires. The rods are plain bars, not threaded, and are fitted with clamps instead of nuts.

*Steel forms, "Engineering News," Oct. 8, 1914.

When the clamp is in place, a setscrew jams the rod against a V-slot in the clamp, securing it rigidly in position (ENGINEERING NEWS, Sept. 10, 1914). Each rod is embedded in a tin tube, so that it can be withdrawn readily, the holes being then packed with stiff cement grout at each end.

The retaining walls are built in alternate blocks of 35 ft., with the traveling forms. It takes about 6 hours to fill the form, which is then left in place about 15 hours. It then takes about 20 hours to release the traveling forms, move 70 ft. forward and adjust them and the sheeting ready for the concrete. The use of the traveling forms has enabled the work to be done in about 25% of the time required with the ordinary forms (from the building to the removal of the form), and at about 50% of the cost (including erecting, pouring and dismantling).

The deck slabs for subway spans were hauled to the site on flat cars at the level of the bridge floor and set in place by wrecking cranes or locomotive cranes. The steel girders of the 79th St. span (weighing about 44 tons) were shipped complete and hoisted into place by locomotive cranes standing on the tracks below; these cranes also placed the floor-beams. All the steelwork of this bridge is encased in concrete, applied by the cement gun.

The filling between the retaining walls was done with trains of 40-ton ballast cars having side doors hinged at the top, the material being plowed off by a rapid unloader. The width of right-of-way is 100 ft., and practically all the work is kept within these limits.

The work-train service is handled by a special trainmaster who is a member of the construction organization, and works in consultation with the operating trainmaster and the dispatcher. The trains bringing in concrete material and filling are operated on a regular schedule. The construction work has been facilitated also by linking up and rearranging a number of separate sidings and industry tracks so as to form a continuous running track for construction trains, on the east side of the line, from the work to the material and slab yard at 93rd St. Thus all work trains and supply movements can be handled over this section of the line without interference with the main tracks.

For purchasing materials, complete estimates were made in advance so that the purchasing department could get the advantage of purchasing material in bulk. Then requisitions are made in such a way as to keep an ample stock in hand, the purchasing department obtaining the material in quantities as required, drawn against the total amount.

The main or running tracks are shifted as required, but only at long intervals. The shifting of the old track crossing at 79th St. was a difficult matter, as it included interlocking plant. The work was prosecuted first along the east side of the right-of-way, leaving the other side free for traffic. It was planned to get this part completed for three tracks over the crossing, with a 1½% run-off from the south end to the present level at 85th St. Then traffic was diverted to these three elevated tracks, and the construction work shifted to the west side of the right-of-way. The traffic over the crossing was controlled by hand-operated switches and signals during construction.

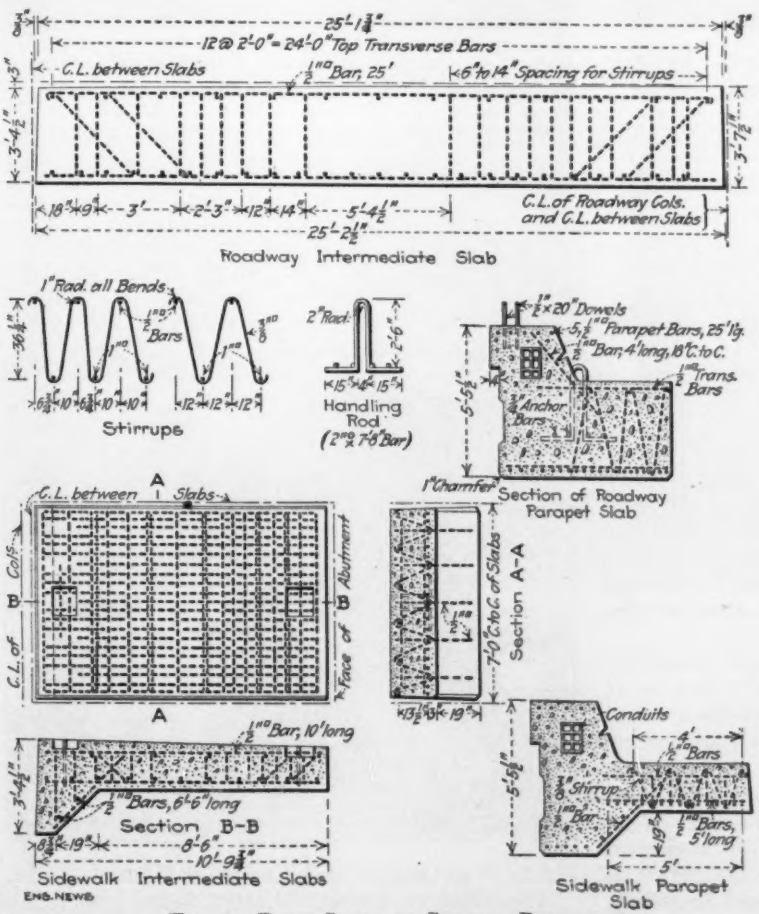


FIG. 4. DECK SLABS OF SUBWAY BRIDGES

ORGANIZATION

All the work is being done by the railway company, except for the heavy steel erection and cement-gun work at 79th St., which are done by contract. The railway work, however, is not being done by the ordinary force-account method, but by a special organization which handles this one job alone and which estimates and carries on the work as a contracting organization might do. Bids were taken on different sections, and the "organization" bids were always lowest, while the work is being done at a cost below these bids. Part of the saving has been sacrificed, however, in expediting the work. The construction is carried on night and day, and the men receive time and a half for all overtime and holidays.

It is considered, however, that this sacrifice in first cost is well warranted by the advantages of getting the work promptly completed for traffic. In some cases track-elevation work of this kind has been in progress from four to seven years and trains are still being detoured to leave the construction free

of interruption; but in this particular case all traffic is being carried over it (with 400 to 500 trains daily over the crossing), and the work is being pushed with the greatest possible rapidity.

The organization headquarters are at the site, and comprise an office car and an engineers' car. There is also a hospital car, where a company doctor is in attendance daily to attend to minor injuries, and he can be called in emergency, but this work is mainly of the "first-aid" character. This car is operated in connection with the insurance company which insures the workmen under the employers' liability law.

FIG. 5. CONCRETING TRAIN ON TRACK-ELEVATION WORK AT CHICAGO; ROCK ISLAND LINES

(The train consists of a mixer car, with hoist, tower and spouts, and two bin cars. Sand and gravel are delivered to the bins direct from cars on the high trestle. The materials are carried to the mixer car by belt conveyors.)

FIG. 6. BUILDING CONCRETE RETAINING WALLS IN TRAVELING TIMBER FORMS

The office car is an old mail car, remodeled, and has desks for the engineer in charge, office assistant, cost engineer, stenographers, etc. It is equipped with desk, file cases and other requirements, and has coat, wash and toilet rooms, the latter being connected to a city sewer.

The working force is varied as required, and ranges from 200 to 500 men. A special feature of the organization system is that each foreman employs his own assistants, and the assistants employ their gang foremen, while these last employ their own men, the appointments being in each case subject to the approval of the immediate superior. The gang foreman is allowed certain limits of pay for his men, so that he can adapt the pay to the market conditions and can also engage men of more or less skill according to his requirements. These gang foremen make daily reports on blank forms. The laborers are hired by the day, so that the force is continually being adjusted to meet the conditions.

The head of this special organization is Robert H. Ford, Engineer of Track Elevation (who reports to C. A. Morse, Chief Engineer of the Chicago, Rock Island & Pacific Ry.). Directly reporting to Mr. Ford are the general foreman, the chief clerk, the cost engineer, the assistant engineer (who is practically the resident engineer, and under whom are the inspectors and field engineers), and the trainmaster (in charge of work-train service). The general foreman has three assistant foremen, in charge of concrete, erection, and track and filling, respectively. Each is practically in entire charge of his work, the purpose being to give each man authority and responsibility, while the general foreman and engineer of track elevation have oversight of the whole force.

In addition, there are inspecting engineers on the work representing the city and the Chicago & Western Indiana R.R. (which latter pays a share of the cost of the 79th St. crossing). The principal officers are as follows: Engineer of track elevation, R. H. Ford; assistant engineer, C. P. Richardson; cost engineer, J. M. Weir; chief clerk, F. J. Nevins; trainmaster, G. R. Speer; general foreman, T. E. Reilly; assistant general foreman in charge of concrete work, A. L. Greenabaum; assistant general foreman in charge of erection, P. Franz; assistant general foreman in charge of track and filling, C. B. Teller.

COST-ACCOUNTING SYSTEM

A very complete cost-accounting system has been adopted for the purpose not only of keeping record of the unit and total cost, but also to see that the work is

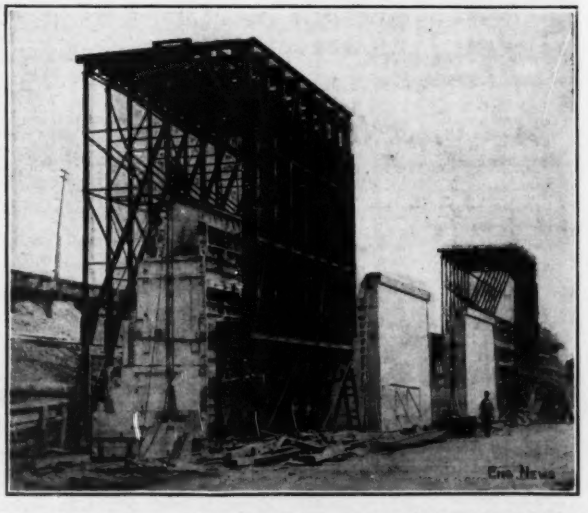


FIG. 7. BUILDING CONCRETE RETAINING WALLS IN 35-FT. SECTIONS IN TRAVELING STEEL FORMS; ROCK ISLAND LINES

done with the highest economy. For this latter purpose the cost engineer spends a good part of his time on the work, studying the methods and conferring with the foremen with a view to devising or suggesting ways in which cost may be reduced and economies effected.

An account is kept (and bulletined) of the amount of work done each week by each gang in the same class of work. In concreting, for instance, there is rivalry between the subforemen to make and maintain good records in this respect, and to do a maximum of work with a minimum of force and payroll. If the cost of any particular item shows an increase, the cost engineer makes an investigation as to the cause and its remedy. He has no authority to order any changes, however, and if the foreman should dissent from the suggestions made, the matter is submitted to the engineer of track elevation.

The reports of the gang foreman are handed to the cost engineer each morning, and he works out the total and unit costs, so that the exact results are known daily.

Concrete-Unit Building Construction at Cedars Rapids*

The large buildings of the Cedars hydro-electric development furnish a good example of the possible fields for the employment of concrete-unit members, molded and cured before placing in the superstructure.

The chief advantage of this scheme from the structural standpoint is the securing of factory conditions for manufacture, inspection and test, giving increased economy and reliability. In this case there were other practical conditions involved which opened the way for the use of this construction. The original plans of the power company called for steel-frame buildings with brick walls, but there were known to be necessarily many troubles and expenses in transportation of the brick since there was no direct railroad connection with the site and carload shipments had to be transported by car ferry on the Soulanges Canal. Straight reinforced-concrete construction was undesirable for the generator house, partly because of the need of using the frame of the building for cranes before concrete members would be sufficiently set. Moreover, the power-house structure was set on a long narrow dam serving also as the turbine casings and generator foundations, so that it was desirable to take as much work off the dam and out of the power house as possible, in order to facilitate the erection of the very heavy machinery. This was possible since the company owned plenty of shore land.

Before the decision was reached to use concrete inset units with the steel frame, the steelwork had already been designed for brick curtain walls, but the details did not require great change. The contractor for the unit construction undertook the erection of the steel, which, however, was furnished by the power company.

Power-House Construction

Walls—The exterior wall panels are 12 in. thick and are formed by two independent concrete slabs, 4 in. thick, with air space between. This provision of double walls was carried throughout the building (even double glazing was used in the windows) and was dictated by the severe winter conditions met with in this locality.

On the steel wall-column angles were riveted where necessary to form slots down which the concrete slabs were lowered. After two opposite slabs had been set (with mortared joints) they were held apart by a piece of 1-in. board slipped into notches in the slab as shown in Fig. 2. This served as a spacer for the slabs and a dam for the grout which was poured between column and slab. A concrete pilaster was cast in place after a wall panel had been completed. The window sills and some of the window jambs were made solid, 12 in. thick.

The crane girders and the main columns were designed to carry two 150-ton traveling cranes. The steelwork over the gates was designed for a single 30-ton crane. The roof trusses were designed for a load of 100 lb. per sq.ft.

Roof—The roof is made up of unit slabs, shown in Fig. 2. Each consists of a 3-in. reinforced-concrete plate, cast with integral beams along all four sides. Each roof-slab is designed as a beam between trusses, the flange reinforcement taking tension and the entire slab taking compression. A 1½-in. space was left between the ends of slabs over the center line of the trusses. These intervals, as well as the smaller ones between the sides of the slabs themselves, were filled with grout. Reinforcing bars 3 ft. long were placed in the spaces between units, passing over the roof trusses as a tie saddle.

Fig. 1. Unit Casting Yard for Cedars Development

*The general design and construction of the Cedars hydro-electric works was presented in "Engineering News," Mar. 25, 1915. The turbines and electrical equipment were described Apr. 1.

Special slabs were required to give the proper roof slope in the gutter next to the walls. Shallow concrete saddles were placed in the gutters to turn the water quickly into the downspouts. The roof covering is a four-ply tarred felt, mopped directly to the concrete and covered with gravel. The downspouts (of copper) were brought inside the building to prevent freezing up.

and four bays 7 ft. long, as shown in Fig. 3. At each end of the building on three floors, a length of 47 ft. was divided into three equal spaces of 15 ft. 8 in. in place of the typical spacing noted, in order to accommodate the transformers; above the third floor typical spacing was used with some of the columns carried by girders.

The load requirements dictated very heavy construction.

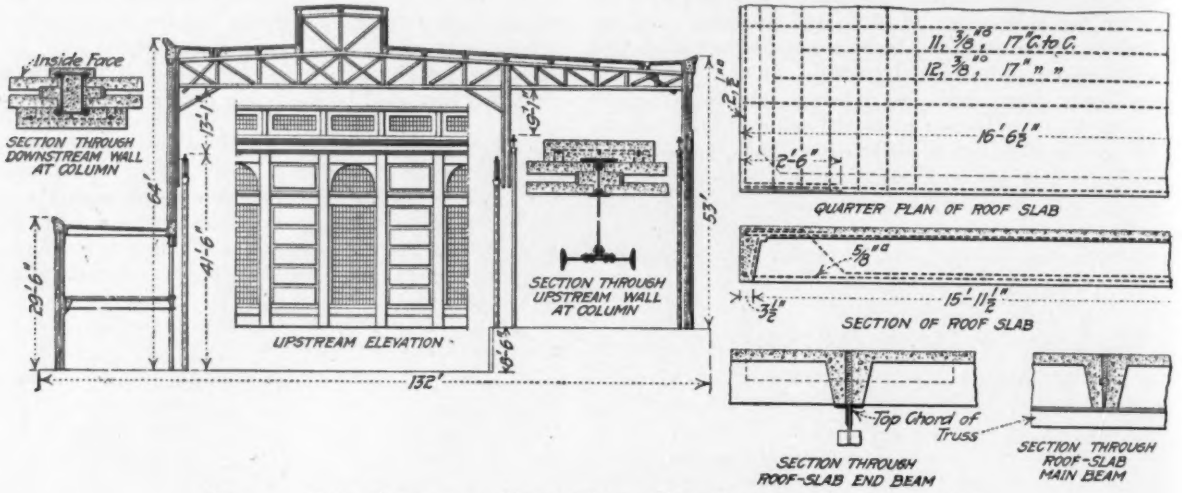


FIG. 2. SOME DETAILS OF POWER-HOUSE UNIT-CONCRETE CONSTRUCTION

ERECTION METHODS—The erection of the power house was started at the north or shore end. Structural steel and formed units were brought to the substructure on flat cars over a trestle track. A structural-steel stiff-leg derrick was mounted on a 60-ft. triangular steel tower. With an 85-ft. boom it was possible to complete the erection for the full width and height of the building, as the derrick was moved back (on rollers) into the clear space.

TRANSFORMER HOUSE

For the shore building, or transformer house, in which is all of the higher-tension apparatus, at some points somewhat different construction was required than used in the power house. This building is entirely of reinforced concrete and has four stories and basement. The building is 228 ft. long, comprising ten bays 20 ft. long

The roof is designed for 100 lb. per sq.ft., and the upper floors for a live load of 250 lb., except at special points; the ground floor has no uniform figure, but is designed, when necessary, to support 75-ton transformers, and to carry standard-gage track on which the transformer trucks run.

FOUNDATIONS—Footings were left on hard clay at 7 ft. below grade and were designed for 4000 lb. per sq.ft. This clay is very hard when dry. Shortly after work on this building had been discontinued for the first winter, the footing trenches filled with water—probably from broken service pipes. It was thought that the heavy overlying snow and ice would prevent the frost from reaching the bottom of the footings, but in the early spring considerable heaving was seen and it was found that water had penetrated the soil so that between the

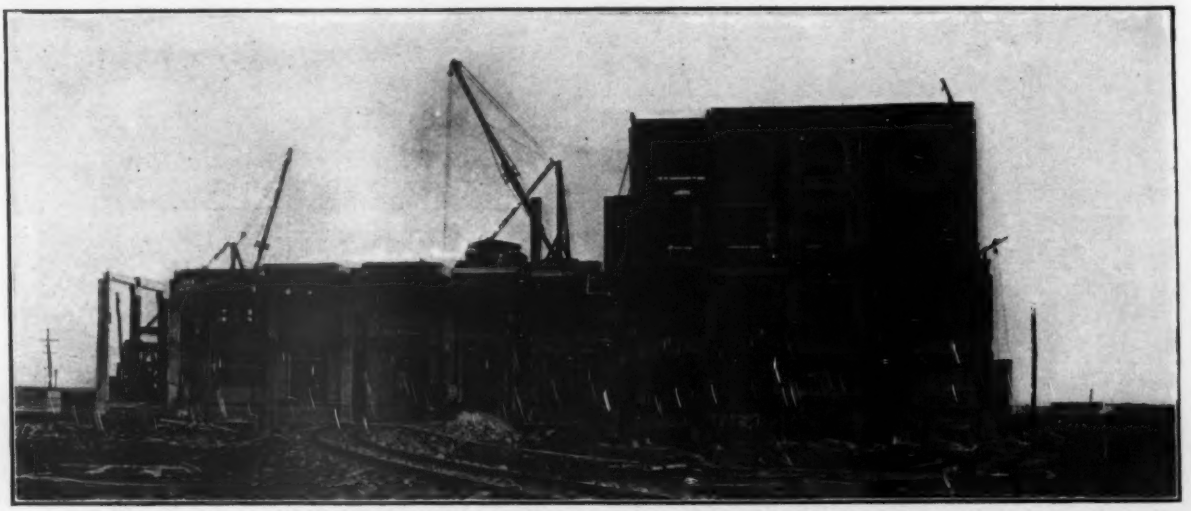


FIG. 3. TRANSFORMER HOUSE AT CEDARS UNDER CONSTRUCTION

strata of clay there were films of ice up to $\frac{1}{16}$ in. thickness. In order to expedite construction, trenches were dug around the footings and the ground thawed out with a steam jet. Cement grout was then worked under the footings. The method was effective, as no settlement occurred after construction, but time would possibly have been gained if the construction work had not been started until the season was further advanced.

WALLS AND FRAME—The exterior walls are 10-in. slabs with a 2-in. air space in the center. These units were not built in two sections like those of the power house but sand cores were placed in the forms while casting, the sand being rattled or washed out after the slabs were pulled. To provide for extension of the transformer house both ends were closed with 6-in. tile walls plastered on both sides. In the basement, side walls are 12 in. thick and of reinforced concrete, but the end walls are of brick for ease in knocking out. Below grade the walls are tarred and drained.

The ground floor showed such lack of duplication of parts and so much special construction that it was decided to build this monolithic, together with the high columns extending up some 35 ft. above it.

The unit girders were made with ledges running the entire length and the floor-slab units rest on the girder ledges for the entire width, with the top of slabs 3 or 4 in. above the top of the girders. Into this grouting space over the girders project bars from both girders and slabs.

All the partitions, with the exception of a 6-in. unit-construction wall, extending the length of the building on the first floor and some tile and glass partitions around the office, are 2-in. solid partitions made by plastering cement mortar on metal lath and steel studs. Partitions between lightning-arrester compartments and transformer stalls are double. Doors to the arrester compartments and in partitions on the high-tension floors are of wire mesh on steel frames.

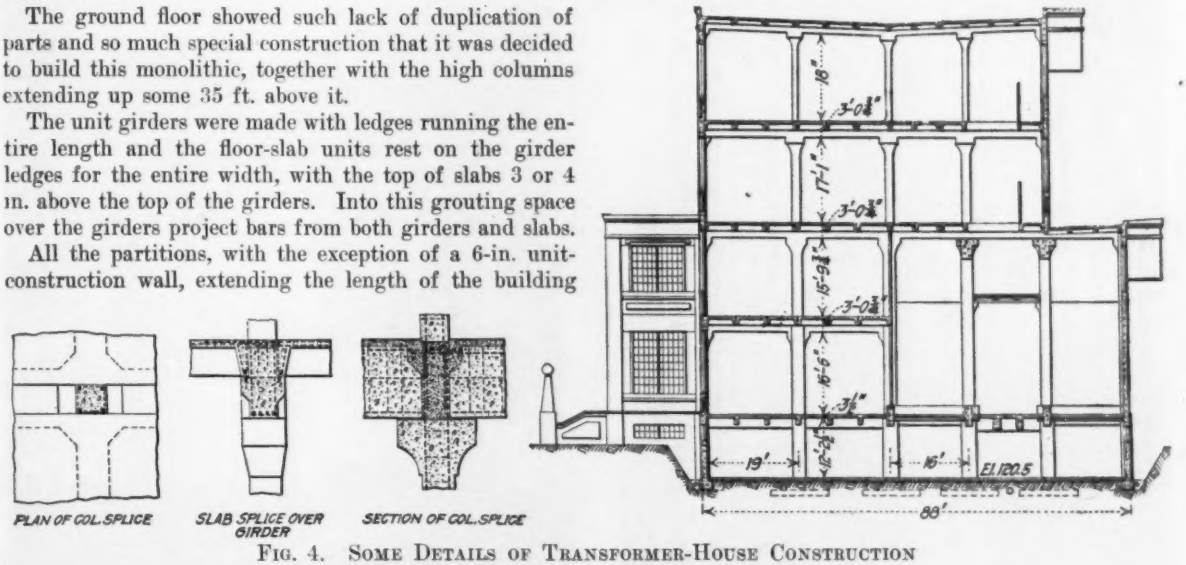


FIG. 4. SOME DETAILS OF TRANSFORMER-HOUSE CONSTRUCTION

ERECTION—The columns, girders, floor slabs and wall slabs, except as specially mentioned, were made in the casting yard and placed in position by derricks. The track was laid along the north side of the building, and units were brought in on flat cars. A steam-operated stiff-leg wood derrick with 75-ft. boom was used on the ground floor and it set all units up to the third-floor level as it moved back. As soon as erection had progressed sufficiently, a second derrick (duplicate of the first) was set up on the third floor and this completed the erection of units to the roof as it moved back. To complete the building, after the first derrick had finished the structure up to the third floor it was set on the roof to dismantle the second derrick and set the few remaining units. Erection proceeded somewhat more slowly than was anticipated, and some delay was introduced by the interference of the high columns with the movements of the derrick.

CASTING YARD

The concrete-unit construction plant was on shore, in a large tract of company land near the Soulanges Canal, with cement-, sand- and stone-storage nearby. Materials were wheeled to the mixer, which discharged into distributing buckets of discharge-control type, and these were handled by a 15-ton locomotive crane with a 50-ft. boom, for which a yard track was built extending the full 400 ft. of the casting yard. The forms were placed on each side of the track for a width equal to the reach of the boom, giving an effective working area of some 36,000 sq.ft. The forms were laid on 4x4-in. sleepers leveled and bedded. The sides of the roof-slab forms were lined with sheet metal for ease in pulling the finished unit. Steel-rod lifting loops were cast into the unit (in the edges of wall slabs and in the corners of roof slabs). After the units were pulled from the forms they were generally taken to the storage yard for further setting.

The number of units and the yardage in the two buildings were as shown in the appended tables:

Roof and floor slabs	1072
Columns	208
Roof and floor girders	279
Sills and lintels	203
Wall slabs	1938
Cornice pieces	37
Jambs	4
Mullions	246
Monitor frames	72
Total	4059
Total cubic yards	5725

An idea of the number of duplicate units is gained from the generator-house roof. This building had ten typical bays 20 ft. long and four 7 ft. long; for each bay there were eight roof slabs.

The power-house steelwork was fabricated by the Phoenix Bridge Co. of Montreal. The contractor for the erection of the steel and completion of the superstructures was the Unit Construction Co., of St. Louis, Mo., J. E. Conzelman, Vice-President and Chief Engineer in direct charge, and J. E. Forwey, Superintendent on the work. As noted in the previous articles, the design and construction of the Cedars Rapids development were under Julian C. Smith, Hydraulic Engineer, and R. M. Wilson, Electrical Engineer, with Fraser, Brace & Co., of New York City, as general contractors.

All-Steel Refuse Incinerator, Roanoke, Va.

By J. C. Woodman*

Roanoke, Va., is a city of some 45,000 inhabitants. For years the city dumped its refuse on vacant land and covered it with dirt. This system was expensive and unsatisfactory. Many injunction suits were brought against the city in 1913 by citizens who claimed that they were damaged by the presence of these dumps. It was impossible to find any new dumping ground that would not meet with opposition, and the matter of purchasing an incinerating plant was put in the hands of a committee.

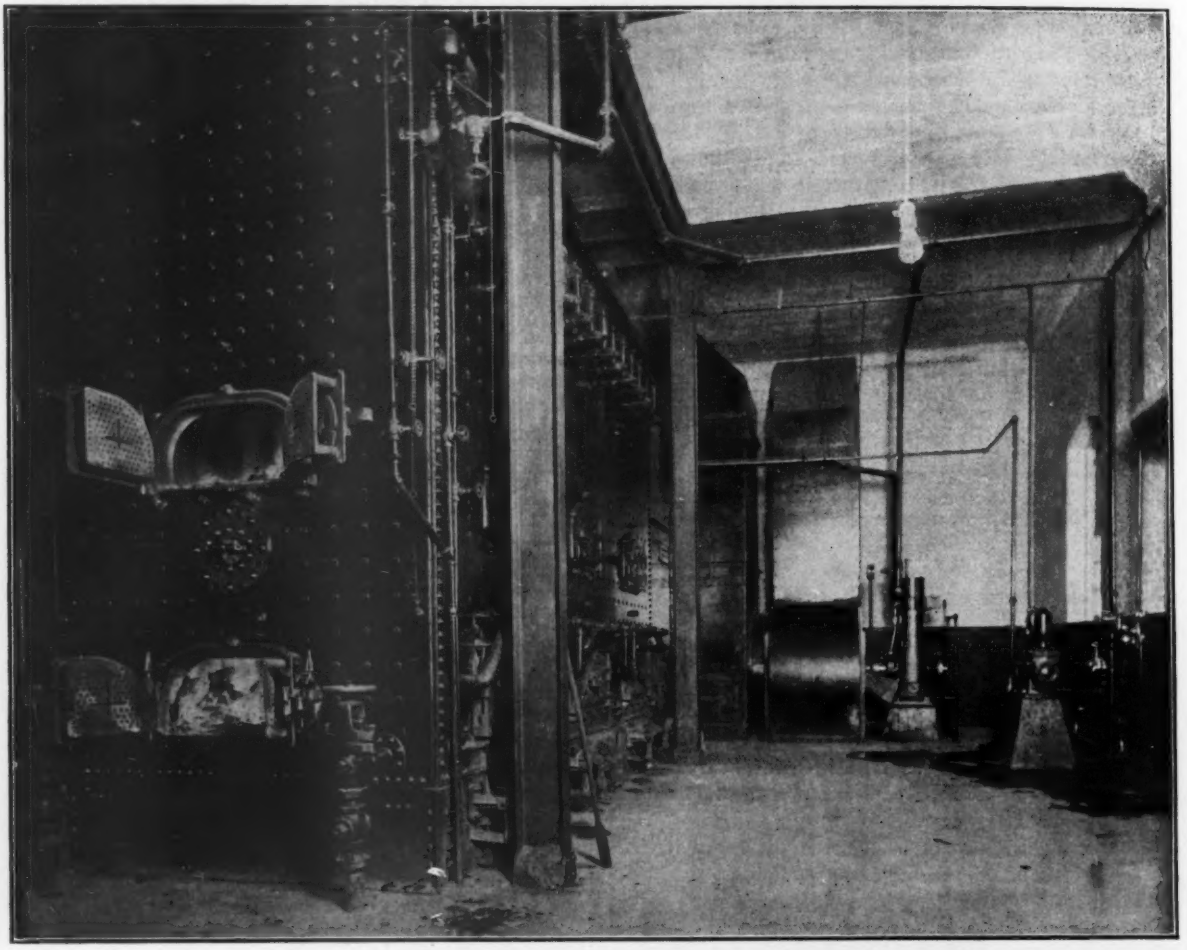


Fig. 1. View of Exterior of Furnace and Operating Floor, Decarie Refuse Incinerator, Roanoke, Va.

Early in June, 1914, an investigating committee submitted a report to the city council, recommending the purchase of a 50-ton Decarie incinerating plant. This report was adopted by the council and in July, 1914, a contract was entered into with the Decarie Incinerator Co., of Minneapolis, Minn., for a 50-ton incinerator at a cost of $41,350. The plant was completed Dec. 15 and accepted by the city Jan. 4, 1915.

General Description

The incinerator is located in the southwestern part of Roanoke, on land owned by the city, and on which the city stables and workshop are also located. A residence section of the city is adjacent to the plant, some of the houses being as near as 200 ft.

A reinforced-concrete driveway approach, 18 ft. wide by 283 ft. long, leads to the upper floor of the building, where all the material is dumped directly from the wagons, through the two charging holes, into the furnace. This approach affords excellent shed room for the storage of equipment or lumber.

No storage bins are necessary in this type of plant, for all refuse is dumped, immediately upon arrival, into the furnace. The furnace is capable of holding 16 cu.yd. of refuse at one time, and by coöperation on the part of the collecting forces the plant at no time will require the teams to wait to discharge their loads.

All refuse, upon being charged, drops through automatic closing doors into a basket-grate formed of water-tube pipes.

The most distinctive feature of this plant is the furnace (Figs. 1 and 2), which is of the all-steel water-jacketed type, having no brickwork whatever to fall down or require repairing. The furnace is 6⅔x15⅔ ft. in plan, giving 90 sq.ft. of grate surface, and 12 ft. high. It is made with a 4-in. water space on all four sides, together with a 2-ft. steam space at the top. The plates are of ⅜-in. steel and the mud ring is of the U-shaped design, made of

*Engineer, Stacy-Bates Co., McKnight Building, Minneapolis, Minn.

½-in. steel plate. The whole furnace is stayed throughout to stand a working pressure of 100 lb. per sq.in. A 10-in. longitudinal header, made of extra-heavy pipe, connects the two end water spaces. From this header the

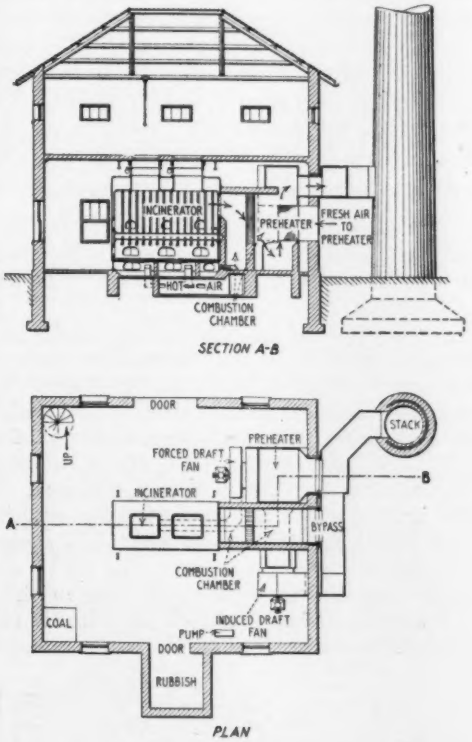


FIG. 2. GENERAL ARRANGEMENT OF PLANT

2-in. extra-heavy basket pipes branch out every 12 in. to form the water-cooled basket-grate that holds the refuse about 2 ft. above the fire on the lower shaking grates. In this way the flames have free access to all portions of the newly charged material without obstructing the draft or deadening the fire in any way. The wetter material becomes dried and partially burns while in this upper grate and then drops of its own accord down on to the lower grate for final combustion.

The gases of combustion pass out into a brick-lined, steel-incased combustion chamber, where fresh oxygen from the forced-draft fan is introduced and thoroughly mixed with them before they go on to the chimney. It is in this chamber that the heat and fresh hot air completely burn the gases and destroy all the odors. (Fig. 2 shows the arrangement of this equipment and the course of the gases.) After leaving this chamber through an opening on the side, the gases pass up through the tubes of the preheater, or regenerator, and are then drawn overhead from the preheater through the induced-draft fan and discharged directly into the chimney.

The preheater is made from 561 2½-in. boiler tubes expanded into ⅜-in. steel plate heads. The air supply for the forced-draft fan is drawn through the preheater around the outside of these tubes, becoming heated to 300° or 400° before entering into the ashpit beneath the grates. The maximum pressure of the forced draft is about 4½ in. The ash pit is divided into three sections and each section has a blast-controlling nozzle so that any section of the grate may be cleaned independently of the others. The hot forced draft is the one essential item that enables the material to be consumed without the addition of fuel.

The forced-draft fan is a No. 110 special American Blower Co. fan and the induced-draft fan is a No. 8 Sirocco, made by the same company. Both fans are steam driven, the steam being generated by the burning refuse in the water-jacketed furnace. The available evaporation per pound of refuse consumed, without the installation of any boiler equipment, is 1.6 lb. of water. The chimney was built by the M. W. Kellogg Co., of reinforced concrete, and is 5 ft. in diameter at the top and 125 ft. high, lined with firebrick.

All the equipment is housed in a substantial brick building of fireproof construction, with an asbestos shingle roof on steel trusses. The floors of the building are of con-

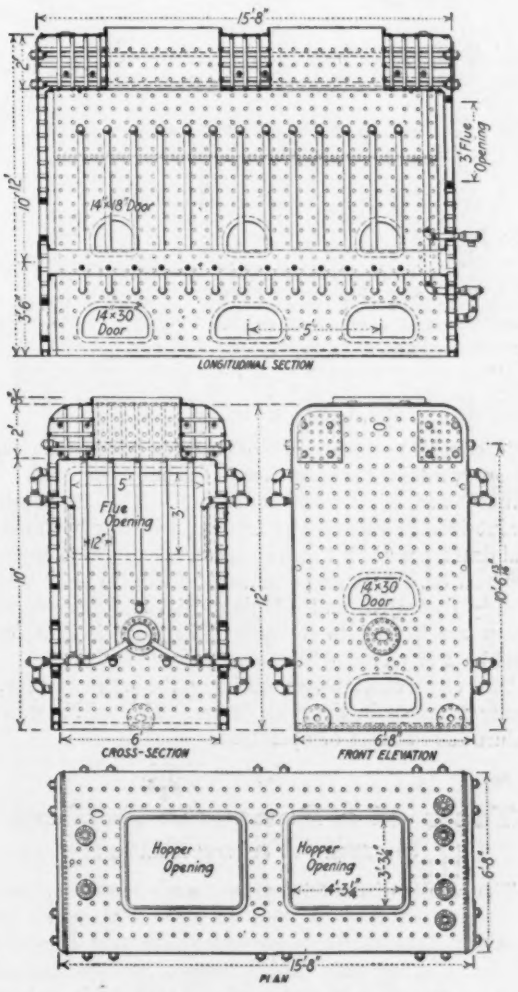


FIG. 3. DETAILS OF FURNACE, DECARIE REFUSE INCINERATOR, ROANOKE, VA.

crete, sloped to the drains. Toilet and locker rooms are provided. Electric lights and hot and cold water, as well as steam-hose connections, are provided on both floors.

CONTRACT GUARANTEES

The contract called for burning completely 50 tons of refuse in 24 hours, without the escape of odors or obnoxious gases from the building or chimney. The cost of incineration was to be not more than 50c. per ton for all op-

erating expenses. This guarantee was made on no stipulated mixture of refuse, as the builders agreed to take the material "as created by the city from day to day." This cost of operation was based on a stipulated maximum cost of labor of 25c. per hr. These guarantees were fully covered by a bond given the city by the builders.

The following tabulations of the tests, made under the existing conditions at a time of year when the refuse in Roanoke is of the worst, will indicate the results obtained:

TEST OF DECARIE INCINERATOR PLANT, ROANOKE, VA.
(Rated capacity 50 tons in 24 hr.)

	No. of Test		
	1	2	3
Date	Dec. 17	Dec. 19	Jan. 4*
Weather	Clear	Rain	Clear
Duration, hr.	10.25	8.5	10.5
Mixed refuse consumed, lb.	47,925	46,980	55,470
Horses and animals consumed, lb.	620	....	....
Total material destroyed, lb.	48,545	46,980	55,470
Per cent. of ashes and incombustible in refuse received (approx.)	60	25	25
Refuse burned per hour, tons	2.367	2.763	2.641
Equiv. incineration in 24 hr., tons	56.8	66.31	63.38
Equiv. incineration per sq.ft. grate per hr., lb.	52.6	61.4	58.7
Labor cost, as per terms of contract	$7.17	$6.31	$6.41
Labor cost per ton as per contract	29.5c.	26.9c.	23.1c.
Actual labor cost as operated by city	$9.00	$8.25	$6.96
Actual labor cost per ton	37.1c.	35.1c.	25.1c.
Cost of water evaporated at 8c. per M. gal.	1.23	1.04	1.28
Cost of fuel required, wood to kindle fire only	$0.50	$0.50	$0.25
Fuel cost per ton of refuse	2.1c.	2.1c.	0.9c.
Total cost of operation per ton as per contract	31.6c.	29.0c.	24.0c.
Total actual cost of operation per ton	44.3c.	41.6c.	30.6c.
Guaranteed cost operation (including labor and fuel) per ton	50c.	50c.	50c.
Condition and amount of smoke	light white	light white	light white
Per cent. combustible in clinker	2.40	2.36	2.31

*This test was made with the regular crew of the city, after the plant had been left for it to operate, following company's test of Dec. 19. Average cost of operation by city from Dec. 19, 1914, to Jan. 4, 1915, was 39c. per ton.

The small percentage of combustible left in the clinker, as analyzed by the city chemist, should be noted. The clinker was of a vitreous nature. The temperatures maintained in this type of furnace, with the hot mechanical draft, are at all times high enough to obtain complete combustion. A large percentage of the tin cans is completely destroyed, and all glass, dirt and incombustible refuse are fused into clinkers. The plant is being operated by a city crew of three men.

The city contemplates utilizing the excess steam for power purposes in the near future. This will materially reduce the net cost of operation.

Tests of a Ransome Drifting-Sand Water Filter

The results of the 36-day trial made some months ago of the 500,000-Imp.-gal. test unit of the Ransome drifting-sand filter at Toronto, Ont., have just been made available. They have double interest because they show the bacterial and physical results given by this novel and unique type of gravity mechanical filter and because, on the strength of these tests and a few small working installations, Toronto shortly afterward awarded a contract for a 60,000,000-Imp.-gal. plant of this type instead of adding to its 32,000,000-Imp.-gal. slow sand filters built only a few years ago.*

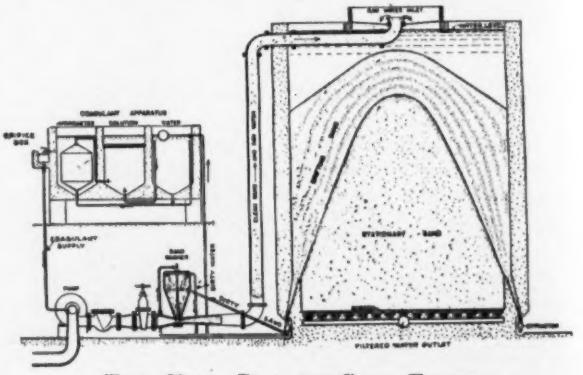
The unique feature of the drifting-sand filter is the continuous washing and renewal of the surface of the filter sand. By arrangements described later, floating or drifting sand is being continually removed from the many-heaped surface of a filter bed of unusual depth, washed in untreated water and then returned to the surface of the bed by the influent raw water, to which a coagulant is added enroute. At long intervals, for a mechanical filter, the whole body of sand is back-washed. There is neither pre-sedimentation nor pre-coagulation. A noteworthy accessory feature is the coagulant-dosing plant.

Description of Test Plant

The test plant at Toronto was located at the old West Toronto pumping station, where sewage-polluted, and frequently muddy, water could be taken from Humber Bay, Lake Ontario.

The arrangement of the filter and accessories is shown by the accompanying sketch. If the filter surface were level its area would be 143 sq.ft., or 0.0033 acre. The estimated area of the "stationary cone" is 306 sq.ft. The depth of stationary sand ranges from 13 ft. to 2 ft. 2 in. and of drifting sand from 10 to 2½ ft. A local screened and washed sand, largely of limestone, was used. It had an effective size of 0.4 mm., a uniformity coefficient of 2.7, and passed through a sieve with eight meshes per linear inch.

The drifting sand passes down the cone and into and through a series of hopper-shaped ports in the circular base of the filter wall. The tops of these hoppers are 2½



Test Unit, Drifting-Sand Filter

ft. above the underdrains, and their bottoms terminate in "extractor outlets" provided with glass "inspection tubes" and also with cocks, which may "be adjusted to suit the character of the water being filtered." From the outlets the drifting sand is discharged into a circular collecting main and carried by the washwater to the sand washer. The latter is described in the report in the following manner:

SAND WASHER—The sand washer consists of a flat-sided hopper-like vessel with glass windows at the flat sides through which the action can be observed. The sand washer is attached to the filter-supply pipe immediately over a constriction in it something like the form of a Venturi-meter tube. An opening in the bottom of the sand washer lets the clean sand through into the passing water, which conveys

*A three-column article on this contract award—giving the competitive bids, describing the drifting-sand filter, giving its history, the engineers connected with it and a list of installations to date, and also summarizing very briefly the Toronto test—appeared on pp. 1446-48 of "Engineering News" of June 25, 1914. The contract was awarded to John ver Mehr Engineering Co., Ltd., Confederation Life Building, Toronto, Ont., and Wm. Cowlin & Son on a joint bid. The first-named company controls the drifting-sand filter in Canada, and installed the Toronto test plant. The tests were conducted by Dr. Geo. G. Nasmith, Director of Laboratories, Toronto Department of Health. The report on the tests is by Dr. Nasmith and Dr. Fred Adams, who is Dr. Nasmith's first assistant. Dr. Chas. J. Hastings is Medical Officer of Health and R. C. Harris is Commissioner of Public Works of Toronto.

it to the filter and maintains the conical heap of sand in it. The pressure in the pipe at the constriction is very much less than on the filter. This difference is sufficient to draw the loose or drifting sand charged with dirt out of the filter into and through the sand washer and into the supply pipe, and so maintain the continuous motion of the sand while the filter is in operation. The sand, which by this time has become freed, but not separated, from the dirt, is led into the sand washer at about mid-height. The weight of the sand carries it to the bottom of the washer and the dirty water escapes at the top. Near the bottom of the washer is a jet of water which carries the sand downward into the supply pipe, displacing the dirty water upward in so doing.

COAGULANT APPARATUS—The sulphate of alumina used as a coagulant was prepared and admitted to the raw water in the volume and of the strength desired by means of three paraffin-coated reinforced-concrete tanks, a float in one of the tanks, and an orifice box with changeable orifices—all as shown in the left-hand portion of the sketch. To quote again from the report:

The first tank contains water supplied from the pumping main and maintained at a constant level by a ballcock. The water from this tank may pass either direct to the bottom of the solution tank or through the needle valve at the bottom of the hydrometer tank.

Aluminum sulphate is fed in to the upper compartment of the solution tank in sufficient quantity to maintain the solution at any convenient amount in excess of 1½%. The solution passes from this tank into the hydrometer tank through a needle valve at the top of the hydrometer.

Thus the hydrometer tank is supplied either with solution at the top or water at the bottom, depending upon whether the hydrometer is up or down. The hydrometer, which consists of an oak barrel with conical ends of concrete, provided with brass adjustable needle-valve points, is weighted so as to just float in a solution containing 1½% of aluminum sulphate. The heavier solution coming in at the top commences to travel downward, producing circulation, and, conversely, the lighter coming in to the bottom commences to travel upward, again producing circulation, with the result that stratification at the orifice cannot be detected. The solution, before passing to the suction pipe, was further diluted about 200%, so that when entering the raw water the solution was of about ½%.

SUMMARY OF TESTS

The 36-day official test was devoted chiefly to bacterial results. Wash-water and loss-of-head observations were also made.

BACTERIAL RESULTS—The first part of the accompanying table is a log of the tests, with the B. coli indications omitted. The last part of the table summarizes the coli tests.

Cultures of B. prodigiosus ranging from 240 to 600 colonies per c.c. were added to the raw water on June 27, after the official test had ended. No prodigiosus appeared in the filtered water.

TURBIDITY—During the official test the raw water ranged from 3 to 45 and the filter effluent was "perfectly clear." Subsequently, finely divided clay was added to the raw water 12 hr. per day for three days, giving turbidities of from 375 to 650. The corresponding filtered water, the report states, was "perfectly clear." During the turbidity test "1½ grains of alum per Imp. gal. of water were used, and the filter was not washed back at any time."

WASHWATER—The raw water which carries the drifting sand from the extractors to the sand washer "amounted in this test to 2% of the total water filtered," and the filtered water used "about every eight days" for back washing the entire sand bed "amounted to somewhat under 1% of the total water filtered," making the total washwater "slightly less than 3% of the total water filtered."

LOSS OF HEAD—When running at 500,000 Imp. gal. per day the loss of head "was about 7 ft." immediately after washing the whole sand bed and 14 ft. just before this washing.

BACTERIAL RESULTS FROM DRIFTING-SAND WATER-FILTER TESTS, TORONTO, ONT.*

(Colonies per c.c. at 37° C. after 24 hr.)

	Raw Water		Filtered Water			
Date, 1913	Total on Standard Agar	Red Colonies on Neutral-Red Bile-Salt Agar	Total on Standard Agar	Red Colonies on Neutral-Red Bile-Salt Agar	Alum Used, Grains per Imp. Gal.	Chlorine used, Parts per Million
May 21	2800		11		¼	
	1800		6			
22	1900		12		¼	
	1650		3			
23	505		2		¼	
	460		2			
24	509		1		¼	
	458		5			
25	363		5		¼	
	560		3			
26	272		2		¼	
	201		0			
27	1325	295	22	0	¼	
	1345	271	31	0		
28	805	180	33	1	¼	
	690	86	17	0		
29	790	144	22	0	¼	
30	95	13	2	0	¼	
	114	20	3	0		
31	353	27	5	0	¼	
	288	33	9	0		
June 1	232	23	3	0	1½	
	295	28	5	0		
2	281	18	7	0	1½	
	203	11	5	0		
3	527	33	5	0	1½	
	1400	65	6	0		
4	198	18	8	0	1½	
	127	27	2	0		
5	251	17	5	0	1½	
	139	10	0	0		
6	1060	39	11	0	1½	
	1095	43	5	0		
7	252	27	6	0	1½	
	202	39	1	1		
8	234	49	2	0	1½	
	223	41	2	0		
9	143	22	3	0	1½	
	89	32	4	0		
10	446	66	1	0	1½	
	492	103	0	0		
11	202	21	1	0	1½	
	362	31	2	0		
12	220	19	3	0	1½	
	244	27	2	0		
13	490	57	0	0	1½	0.3
14	1025	340	4	0	1	0.3
	515	67	0	0		0.3
16	1540	174	0	0	1	0.5
	1080	187	0	0	1	0.3
	670	85	2	0		0.25
17	1010	75	3	1	1	0.2
	550	49	0	0		0.0
18	3850	270	22	0	1	
	3600	245	21	0		
20	21000	730	141	19	1	
	20000	840	115	12		
21	1380	126	53	1	1	
	1170	101	66	1		
22	111	91	38	0	1½	
	244	54	33	0		
23	415	72	71	0	1½	
24	186	53	8	0	1½	
25	159	38	4	1	1½	
26	4550	144	18	1	1½	
27	380	33	0	0	1½	
Average	1365	13	106	0.7		
per cent. removed			90.0	99.3		

*As a rule, two samples of raw and two of filtered water were collected daily. Counts on gelatin after three days at 20° C. were made, but were discarded, for this reason: In the city mains and reservoirs from which the water for backwashing was taken a bacillus multiplies at times which gives numerous yellow colonies on acid gelatin. This bacillus apparently multiplied in the filter. This yellow organism grows on acid gelatin, but does not grow on neutral gelatin, such as used as a routine by Dr. Houston of London; nor does it grow on agar in the warm incubator, and is of no sanitary significance. The gelatin counts were consequently disregarded.

AVERAGE B. COLI RESULTS

(In Lactose Bile at 37° C. after 48 Hr.)

	C.C. of Raw Water					C.C. of Filtered Water				
	50	10	1	1/10	1/100	50	10	1	1/10	1/100
Total tests..	58	66	66	66	66	60	66	66	66	66
Positive....	58	66	66	47	11	38	22	7	1	0
Negative...	0	0	0	19	55	22	44	59	65	66
% Positive.	100	100	100	71	17	63	33	10	2	0
% Negative	0	0	0	29	83	37	67	90	98	100

Average number of B. Coli in 50 c.c. of raw water..... 1120
Average number of B. Coli in 50 c.c. of filtered water..... 14
Average number of B. Coli in 50 c.c. per cent. removed..... 98.7

One Meter Reader and One Bill, instead of three readers and three bills, for water, gas and electric-light service, is the recently adopted practice of Bonn, Germany, under municipal ownership.

Field and Office

Sizing-Up and Bidding on a Contract in Dull Times

By One of the Contractors Who Bid

The job-hunger of contractors and the opportunities for getting construction work done cheaply at the present time are illustrated in the recent bidding on a contract covering the construction of a railroad bridge. The railroad sent out invitations to about 40 contractors and several more asked for the privilege of bidding. In all 38 bids came in, including mine; the contractors ranged in residence from Maine to Missouri and from New York to Georgia.

BIDS FOR THE DOUBLE-TRACK CONCRETE ARCH OF THE CUMBERLAND VALLEY R.R. CO. BRIDGE OVER SUSQUEHANNA RIVER

Firm	* Concrete Masonry, Class A, Per Cu. Yd., Price	Waterproofing, 167,000 Sq. Ft., Price	Conduit, 7,400 Lin. Ft., Price	Manhole Frames and Covers, 21 Sets, Price	Removing Old Masonry, 100 Cu. Yd. (Est.), Price	Total
Robert Grace Contracting Co.	$4 75	$0 023	$3 00	$30 00	$1 00	$790,290
E. Farrell	4 75	0 02	0 05	6 00	2 00	799,964
W. W. Braley & Co.	4 77	0 03	0 05	15 00	20 00	779,425
Booth & Flinn, Ltd.	5 03	0 025	0 05	12 00	5 00	802,201
T. L. Eaves	5 13	0 03	0 20	10 00	2 00	788,695
O'Hagan & Lake, Inc.	5 75	0 02	0 05	6 00	2 00	890,000
Hardaway Contracting Co.	5 95	0 06	0 05	20 00	10 00	846,700
MacArthur Bros. Co.	5 75	0 04	0 05	15 00	2 00	812,054
Walter P. Patterson	6 00	0 03	0 20	10 00	2 00	830,770
Charles McDermott Co.	5 93	0 03	0 05	10 00	2 00	829,425
Thomas Sheahan	6 00	0 025	0 10	12 00	1 00	842,972
The Thayer Contracting Co.	6 00	0 025	0 10	25 00	1 00	864,900
The H. C. Brooks Co., Inc.	5 90	0 10	0 50	30 00	2 00	848,000
Frazer, Brace & Co.	6 10	0 025	0 05	30 00	1 00	856,070
M. S. Roebuck, Inc.	6 20	0 03	0 05	30 00	20 00	861,500
Patterson-Murray Co.	6 20	0 04	0 05	30 00	3 00	872,000
The National Foundation & Engineering Co.	6 00	0 12	0 00	30 00	1 00	875,370
H. Desbrugger Construction Co.	6 00	0 025	0 05	30 00	1 00	876,111
John Gill & Co.	6 00	0 05	0 10	15 00	1 00	896,625
Peters & Rogers Construction Co.	6 00	0 025	0 10	25 00	1 00	899,710
Smith-McCormick Co.	6 04	0 03	0 10	30 00	1 00	899,190
The Dunn-McCarthy Co.	6 11	0 05	0 10	15 00	1 00	899,632
Sanford & Brooks Co.	6 11	0 05	0 10	25 00	1 00	905,985
Trowbant & Dennis Co., Inc.	6 11	0 05	0 00	25 00	2 00	914,025
The Snare & Triest Co.	7 20	0 03	0 00	25 00	20 00	919,000
Merrill-Ryckman Co.	7 20	0 05	0 25	25 00	1 00	919,240
Sparks & Evans	7 20	0 015	0 10	25 00	2 00	920,162
F. E. Brush Contracting Co.	7 20	0 02	0 10	6 00	1 00	920,923
Arthur McMullen	7 20	0 04	0 75	30 00	20 00	921,330
Dugdale Contracting Co.	7 20	0 04	0 75	30 00	2 00	922,200
Raymond Concrete Pile Co.	7 67	0 04	0 10	25 00	2 00	940,345
James McGraw Co.	7 70	0 03	0 10	30 00	2 00	942,791
F. M. Talbot Co.	7 70	0 025	0 00	25 00	2 00	942,918
Healy, McCarthy, Burke Co.	7 70	0 03	0 05	30 00	2 00	946,151
The Johnson-Gifford Co.	8 15	0 025	0 10	30 00	2 00	990,000
A. L. Anderson & Bros., Inc.	8 15	0 03	0 05	30 00	2 00	990,040
Geo. H. Carter Construction Corporation	8 20	0 10	0 05	30 00	6 00	1,016,139
L. H. Pasbi & Son	11 50	0 035	0 05	30 00	2 00	1,357,000

* Concrete, reinforcing steel and anchor bolts to be furnished on cars, at site of bridge by the railroad company.

Bidders and Prices on New C. V. R.R. Bridge at Harrisburg, Penn.

(Reproduced from "Engineering News," Mar. 25, 1915.)

The work to be let was the Cumberland Valley R.R.* bridge across the Susquehanna River at Harrisburg, Penn. The structure is to be about 3995 ft. long, have reinforced-concrete arches and a double-track roadway, of which one trackway is to be built in one season (about 200 working days), and take about 40,000 yd. of concrete.

There are 23 old stone piers that will be incased in concrete 1½ to 3 ft. thick and heavily reinforced. A new pier is to be constructed between each pair of old ones. The new foundations are now in to several inches above low-water level, having been placed last year by the railroad company's own forces in a remarkably short time and under very favorable river and climatic conditions. There will be 21 arches having a span of 77 ft., 22 arches of 74½ ft., and 3 of 69 ft., with 20-ft. rise. The different-sized spans and radii are so mixed that the arch-centers of one size cannot be shifted to the next. This results from the fact that the old stone piers were not located a uniform distance apart.

The stone piers were built for double track, but only one track was laid. The steel structure carrying this track was recently shifted from the center to one side of the piers to make room for the first half width of the new concrete bridge. This new track will carry all traffic while the second track is being laid.

Of the 4000-ft. length about 3600 ft. is over water, on low stage from 18 in. to 5 ft. deep; but a dam, nearly completed, a little below the bridge will raise the water some. Stopping shifting sand, however, may cause the anthracite-coal washings to raise the bottom of the river. The average ultimate depth will probably be about 2 ft. in one-half of the river. All that can be counted on in the other half is 3 to 5 ft., except around old piers where there are riprap and sand bars.

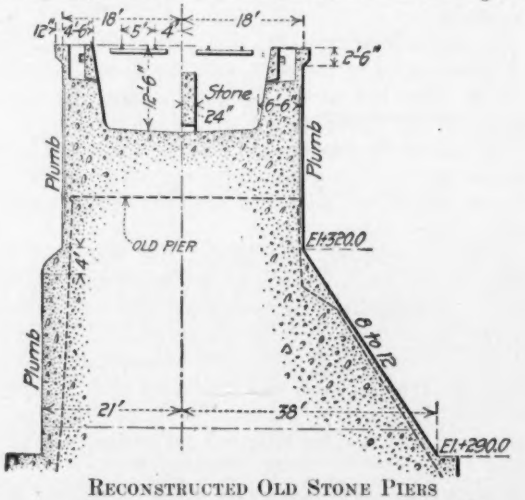
Working Methods Proposed

Proposed methods of doing the work were possibly as numerous as there were contractors.

The river has a range of 15 ft. above the low stage previously mentioned, and floods are likely to come any month of the year, though the summer is usually counted safe from floods. On low water a floating equipment would be difficult to manage.

To build a working trestle, low enough to permit floods to wash over it and not wash it away, would interfere with the coal fleet (suction dredges pump the river bottom for anthracite-coal washings from mines, and get a large tonnage which is sold to local steam plants) unless drawspans were maintained. To build high enough to clear the coal fleet would possibly mean loss in case of high water—and surely if left in for winter ice to hit it.

On the Harrisburg end a street runs under the old bridge, and between the first and second piers is the river bank with sidewalks and steps leading to the water. This, no doubt, must be kept open for travel. In the center of the river is an island about 350 ft. wide, which is a public playground. The railroad claims a 60-ft. right-of-



Reconstructed Old Stone Piers

way, and the city park commissioner will permit contractors to install plant on a portion of this island. On the Lemoyne side of the river there are railroad and trolley tracks, and lack of room generally.

Should cableways be used for handling material the long water-spans and the restricted space at bridge ends will necessitate the use of high towers to allow buckets to clear the bridge, which will be about 50 ft. above water.

*Thos. B. Kennedy, Engineer, Crosby Tappan, Engineer of Construction, C. V. R.R.

Furthermore, the cableway would have to be shifted to construct the second half of the bridge.

Train movements on the present structure are all passenger, amounting to 96 in 24 hr. Only twice in a day does a 40-min. cessation in traffic occur. The railroad under certain conditions and at a set price will let material be unloaded by work train from the present structure; but the cost of this service would make the cost of unloading prohibitive.

There is very little fancy work, but variation of sizes of piers will interfere with form duplication.

If steel centers are used to carry the arches they will cost about $1000 for each span-section purchased. They will have to be left in place 30 days. The centers will have to be carried from the foundations on falsework and 45 must be placed in one season. Wood centers might be used with some economy and some risk.

Estimated Cost

Lumber prices ranged from $17 to $22 per M; large size timbers $24 to $30. The area to cover will require a large amount of lumber unless frequent reuse cuts down the quantity.

The quoted price on stone from local dealers after freight and switching charges have been paid was $1 per net ton. There is a quarry nearby where contractors can get crushed stone under certain restrictions and 10c. per yard royalty. Whether, after paying royalty, freight, switching charges, quarry expense, etc., the contractor is ahead of the game would have to be determined after he pays all his bills. Bank sand will cost $1.08 per net ton, delivered on the site. Local river sand is unfit for use.

Labor will cost about 16c. per hr., and carpenters 30 to 40c. per hr.

The railroad furnishes f.o.b. job all cement and reinforcing steel.

About 3000 anchor-bolt holes 18 in. deep will be drilled in the old stone piers; the concrete prices bid include placing steel anchor-bolts in piers, anchor-bolts for railing, and about 1100 ft. of drain tiles. Waterproofing, placing conduits and tearing down old masonry will be extra. The stone backfilling on arches will be put in by the railroad company, as will also the handrail.

There will be 56,000 yd. of concrete (foundations now in place); 600 tons of steel, 7800 ft. of six-duct conduit; 100,000 sq.ft. of waterproofing, and 150 yd. of old stone masonry to be removed.

Low Prices Due to Dull Times

This is possibly the longest (Galveston, Tex., Causeway excepted) concrete-arch railroad bridge in the United States, if not in the world. Yet under ordinary conditions of business in construction lines I doubt if there would have been 15 bidders. Even if it were on land with plenty of room and not a rush job, the prices bid would be higher than those bid under the conditions just described.

In comment upon the prices of the low bidders the following tale may bear repetition. A contractor in the Middle West once obtained a big concrete-bridge job at a very low figure. Upon opening the bids for the next contract of like character this same contractor was found among the high bidders. When asked why this was so he replied: "I have built mine."

A Rock-Sounding Rig

A simple rig which has been used both to obtain the thickness of earth cover over rock and as a drill for blast-holes in soft shale or hardpan has been devised by J. L. Weller, Engineer in Charge of the Welland Canal,

Fig. 1. Rock-Sounding Rig in Use on Welland Canal

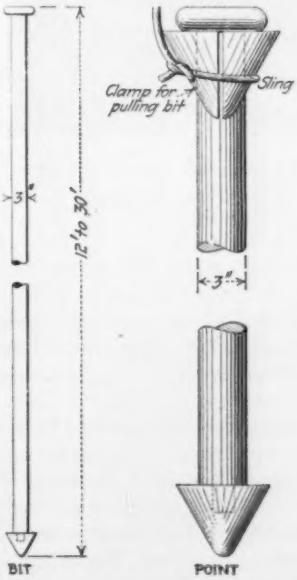


Fig. 2. Bar and Attachments for Rock-Sounding Rig

St. Catharines, Ont., and has been used on the revision work in that canal and on an earthwork contract in Nova Scotia.

The rig essentially is a light piledriver which is used to hammer a long bar into the ground. The piledriver (Fig. 1) is about 25 ft. high and weighs about 200 lb., so that it is easily portable by two or three men. It carries a 135-lb. hammer operated by hand through a single line over a sheave at the top of the leads. The bar is 3 in. in diameter, with an upset head and a tenon point into which fits a driving point of conical shape and slightly larger outside diameter (Fig. 2). New

driving points, at a cost of 2c. apiece, are provided for each operation, the old one being left in the hole. For pulling the bar a clamp in the shape of a bifurcated cone (Fig. 2) is fitted under the head of the bar and the pile-driver rope slung around the clamp. A pull on the rope tends to bind the clamp to the bar and at the same time to pull the bar upward.

The rig was used extensively on the Welland Canal work to locate the rock surface, which lay in a fairly uniform plane beneath the ground surface. The easily portable rig made quite easy and cheap the determination of rock depths at close spacings. On the Nova Scotia job the rig worked ahead of the steamshovel, making blast-holes for small shots to break up the shaly formation.

Replacing Pins in a Railway Bridge under Traffic

The New York Central R.R. West of Buffalo—formerly the Lake Shore & Michigan Southern—crosses the Cuyahoga River at Cleveland, Ohio, by a wrought-iron swing span 300 ft. long built about 25 years ago. A very dense traffic passes over the bridge (until recently about 200 trains every 24 hr.). In spite of its age and heavy traffic,

FIG. 1. BORING OUT THE END-PIN HOLE FOR A LARGER PIN, CUYAHOGA RIVER BRIDGE—FRONT OF BORING RIG

the bridge is in excellent condition for continued service, but one detail recently required reconstruction of an unusual kind. The lower chord of the truss is of rigid type, and the web members are connected to it by pins. The end pins and their pin holes had become so badly worn as to necessitate repairing.

The pin holes were worn oblong vertically, so that there was 5/8 to 7/8 in. vertical movement between the end-post and the lower chord. There were a few minor weaknesses in the floor—for instance, some of the angles connecting the stringers to the floor-beams were partly split at the root. Repair of these defects was simple, however. The important job was the pin replacement. This undertaking consisted in removing the old 5½-in. pins, boring out the holes to 6½ in., and inserting new pins of a larger size.

It was necessary to get the repairs completed before about Apr. 1, the opening of the navigation season, because the method used made it impossible to swing the bridge.

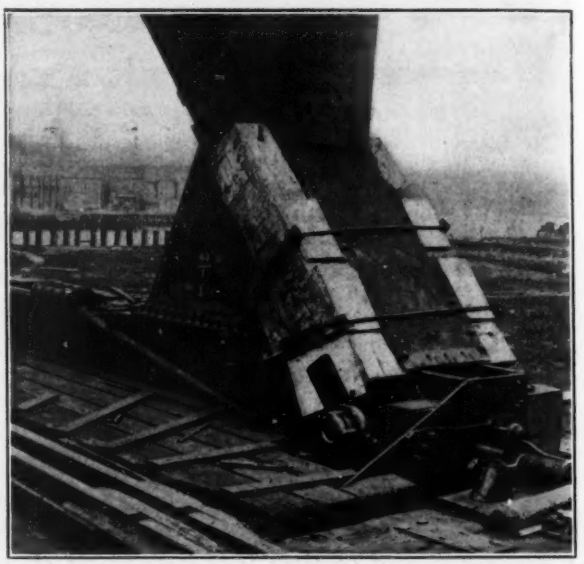


FIG. 2. REAR OF BORING RIG, SHOWING TAIL HOUSING

The fact that the west arm is over dry land simplified the work greatly. The arm could be blocked up on the ground and adjusted to proper height and camber. The members being relieved of stress by the direct support, relatively simple arrangements sufficed to hold the members meeting at the joint under repair. The blocking further insured the true position of the enlarged hole. When the west arm had been repaired, the bridge was swung end for end to bring the east arm over the blocking, and the same procedure was gone through with this arm.

A boring rig was made up (see the photographs herewith) on a frame formed by a pair of housings bolted to timbers fastened on either side of the truss. Adjustable boxes set in the housings carried the boring bar. The cutter head on the boring bar contained two cutters. A feedscrew sunk into the side of the boring bar was manipulated by a handwheel at one end of the shaft. The boring bar was rotated by worm gearing from an air motor. At first a Cleveland Pneumatic Tool Co. No. 4 (four-cylinder) motor was used. When this proved too powerful

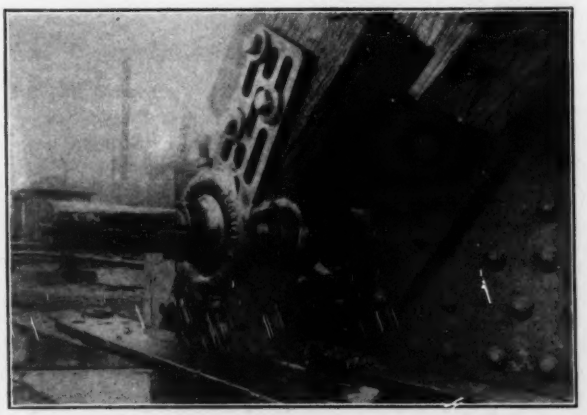


FIG. 3. REAR END OF BORING BAR, FEED SCREW AND BORING HEAD

and ran hot, it was replaced by a Thor No. 9 (two-cylinder) motor, which was used successfully throughout the rest of the work. The pictures show the Cleveland Pneumatic motor in use.

The actual drilling time for three of the holes ranged from 10½ hr. to 12 hr. The entire job, work proceeding night and day, was done between 9 a.m. of Mar. 2 and 2 a.m. of Mar. 6, 1915, out of which time 18 hr. were consumed in moving equipment from hole to hole and swinging the bridge end for end.

Two cuts were taken in each hole. A peculiarity about the metal, as shown by the drilling, was that the first ¼ in. around the hole had become quite hard, about as hard as steel, evidently due to the peening action which produced the enlargement of the holes.

The work was done by the bridge forces of the railway company, under B. R. Leffler, bridge engineer.

Service and Tests of 3-In. Wire Cable

Wire cable over 2 in. in diameter is unusual, and a 3-in. cable is the largest ever made. The first of this size was made for the Spanish-American Iron Co., in 1910, for use in operating an inclined cable railway at the Mayari iron mines in Cuba, where 50-ton cars are lowered in 3-car trains on a maximum grade of 25%. These conditions required a cable having a safe tensile strength of about 150,000 lb. Each cable is about 7900 ft. long and weighs in the neighborhood of 125,000 lb. The most recent of these cables was installed in March of last year.

The cables at the Mayari mines pass over drums 20 ft. in diameter and are carried along the track on manganese-steel rollers, although wood rollers were first used. The newest of the cables, placed a year ago, has four laps around the drum; all previous cables had but three laps.

Service records were kept for the first two 3-in. cables installed, designated as No. 2 and No. 3 cables; No. 1 cable was of 2¾-in. diameter, and its service was only 13½ months. The following tabulation gives the service records of the two 3-in. cables:

	No. 2	No. 3
Placed in service	Jan. 16, 1911	July 3, 1912
Removed	July 1, 1912	Mar. 21, 1914
Time in service	17½ months	20½ months
Number of trips with ore:		
Three-car trips	.....	4
Two-car trips	10,154	5,966
One-car trip	2,194	12,488
Total trips	13,068	17,718
Total ore cars lowered	23,222	22,992
Trips with other cars	.....	408
Tons of ore lowered	1,144,689	1,104,926

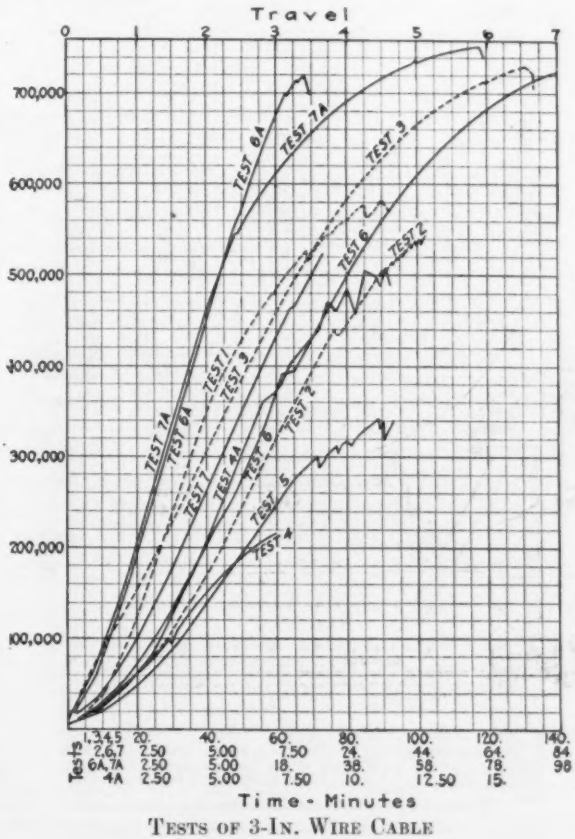
During the period that the No. 3 cable was in service important changes were made, and owing to the conditions of operation this cable received unusually severe treatment. To minimize slippage the amount of cable lubricant was reduced, and as a result the corrosion of inside wires in the samples of No. 3 cable was noticeably greater than in samples of No. 2 cable. The total stretch of No. 2 cable was 32 ft. 3 in., or 0.41% of the total cable length of 7830 ft., and the total stretch of No. 3 cable was 22 ft., or 0.28% of its length of 7980 ft.

Cable No. 3 was made with a 1⅛-in. wire-rope center composed of six strands of 19 wires each about a hemp center. Around the 1⅛-in. center wire rope were six strands of 19 wires each. The wires were plow steel, having a tensile strength of about 225,000 lb. per sq.in.; they varied in size from 0.061 in. to 0.225 in. The steel in the cross-section of the cable No. 3 was 4.1178 sq.in., which gave an assumed tensile strength for the cable of 723,350 pounds.

Strength of Worn Cable

The greatest number of broken wires in a length of 6 in. was 40 out of a total of 72 outside wires. Theoretically 40 broken wires would reduce the tensile strength of the cable about 30%. In addition to this, the wear somewhat reduced the areas of outside wires. Tests of the worn cables showed ultimate strengths of from 47.8% to 70% of the strength of the new cable.

Tests on 14-ft. sections of cable No. 2 gave 582,000 lb. and 594,600 lb. as strengths of used cable and 728,100 lb. as strength of unused cable. Used sections of cable No. 3 developed strengths of 216,200 lb., 342,800 lb., 723,350 lb. (practically uninjured), 719,200 lb. (part of the same specimen as preceding) and 506,000 lb. Tests of unused cable gave strengths as follows: Cable No. 2, 728,100 lb.; cable No. 3, 723,350 lb., and the new cable now in use, No. 4, 751,600 lb.



Tests of 3-In. Wire Cable

The accompanying diagram shows the results of these tests. Tests Nos. 1, 2 and 3 are of cable No. 2, as noted. Tests Nos. 4, 5, 6, 6A and 4A are of cable No. 3, as noted. Tests Nos. 7 and 7A are of unused sections of the new cable No. 4. In test No. 7 one of the yokes broke, which accounts for the low maximum.*

*For further information the reader is referred to a paper by James E. Little, Engineer, Spanish-American Iron Co., Steelton, Penn., in the "Journal of the Engineers Society of Pennsylvania."

Special Features in the New Intake Tunnel at Milwaukee, Wis.

By Lee G. Warren*

The new Linwood Ave. intake tunnel for the water-supply of Milwaukee, Wis., which was described in Engineering News of June 18, 1914, is to be about 4000 ft. long under the bed of Lake Michigan, and is of circular section, 12 ft. inside diameter. It is being driven in shale rock. The contractor is Joseph Hanreddy, of Chicago, Ill. The work is under the general direction of Geo. Staal, City Engineer, and the writer is Resident Engineer, supervising the construction.

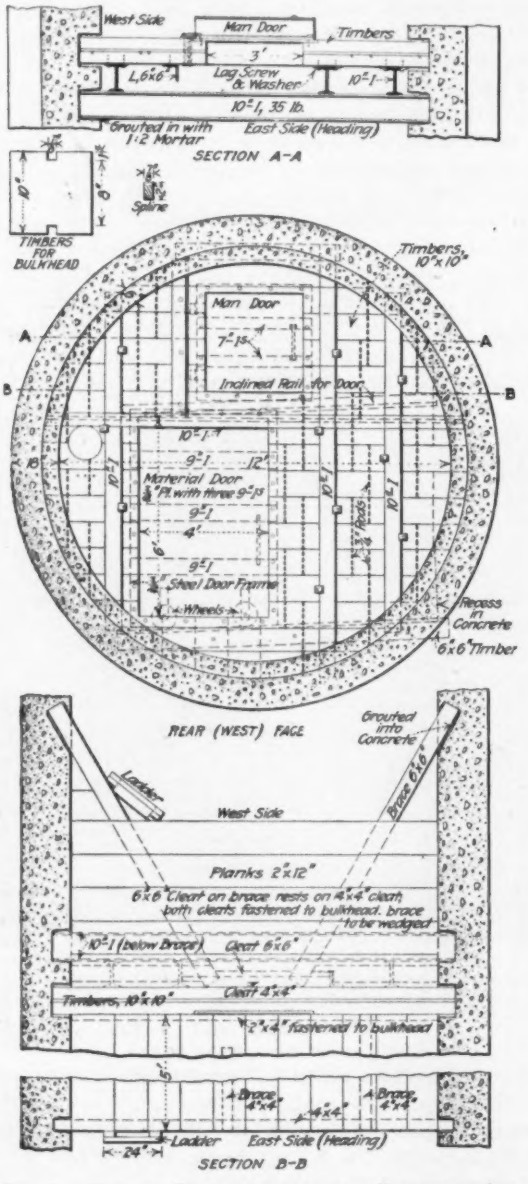


Fig. 1. Safety Bulkhead in the Linwood Ave. Intake Tunnel at Milwaukee, Wis.

*Resident Engineer, Linwood Ave. Intake Tunnel, Milwaukee, Wis.

Safety Bulkhead

Owing to the two serious roof collapses in the tunnel excavation, at stations 28 and 29.09 (Engineering News, Oct. 29 and Nov. 12, 1914), it was decided to install a safety bulkhead, behind which the workmen in the heading might retire if any additional falls should occur. It was feared that a more serious roof collapse might happen, bringing a heavy inflow of water from Lake Michigan and endangering the workmen in the heading.

The safety bulkhead, shown in Fig. 1, was installed at station 28+79. It is composed of 10x10-in. tongued and grooved timbers, reinforced with one 6-in. 20-lb. I-beam and two 10-in. 35-lb. I-beams placed vertically, and one 10-in. I-beam placed horizontally at about the springing line. In the lower half it has a 4x6-ft. material door, and in the upper half a 3x3-ft. man door. Both doors are of ¾-in. steel plate. The material door is supported on two steel wheels, which travel on a steel track, the door sliding laterally. The upper door is hinged, opening toward the heading. A wooden platform, 7 ft. long and extending the entire width of the tunnel, is placed just below the man door, on each side of the bulkhead. The platforms are reached by ladders. The outside perimeter of the bulkhead rests in a 6-in. groove in the concrete lining of the tunnel.

The placing of this bulkhead created an additional air chamber in the tunnel, because every precaution was taken to assure airtightness in the structure. In case of any emergency arising to require the use of this bulkhead, the material door would be closed first. This would stop the rush of any soft material and would temporarily check the water. The laborers in the meantime would have a chance to escape through the upper exit, or man door. Up to the present time, it has not been found necessary to use this bulkhead, but on two occasions everything was prepared to throw it into immediate use.

The costs were as follows: Lumber, $75; steelwork, $200; valves, $70; labor of installing, $155; total $500.

New System of Timbering

Great difficulty had been experienced in maintaining in position the timber sets between stations 27+50 and 32+60. The material encountered in excavating is Hamilton shale, which disintegrates and scales off soon after being exposed to air. This scaling deprived the timber sets of their supports, the side hitches in the shale collapsing. At station 32+61 it was decided to adopt a new timbering method, shown in Fig. 2.

Into the base of each side hitch cut for the roof timber, three 1½-in. holes, 3 ft. deep, were drilled with a pneumatic hand-drill, and then a 1½-in. steel rod, 3 ft. long, was driven into each hole. On the set of three rods was placed a block 2x10x12 in. The round roof timber, of maple, 12 to 14 in. in diameter, was then laid on the small blocks resting on the rods. Finally, slanting braces of maple 6x6 in. were fitted to the roof timbers, these braces resting in hitches that were cut in the side of the excavation. Frequently the supporting rods are placed at the hitches for the slanting braces instead of at those for the roof timbers.

The timber sets, starting at station 32+61, are being placed 2 ft. c. to c. Since inaugurating this new timbering method no roof timbers have fallen and no roof collapses have occurred. Numerous instances have oc-

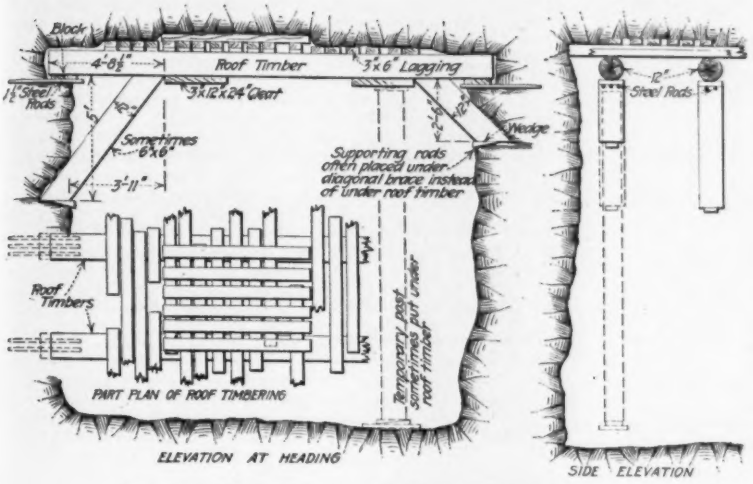
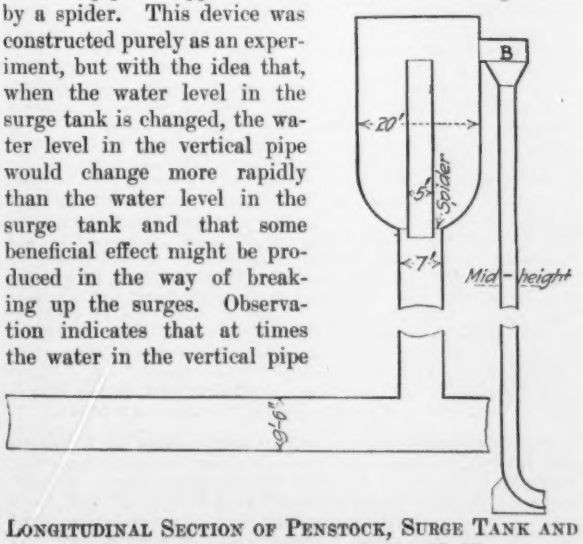


FIG. 2. NEW METHOD OF SUPPORTING THE ROOF TIMBERING IN SHALE; MILWAUKEE AVE. INTAKE TUNNEL

curred where the excavation sides have scaled off beneath the hitches, leaving the steel rods entirely exposed for 1½ ft., but they showed no signs of failing.

Results with a Novel Surge Tank

A surge tank containing an unusual feature was installed at the Baird-Violet Falls plant of the Northern New York Utilities Co. on Beaver River in connection with the recent development of that power. The general arrangement and dimensions are shown in the accompanying sketch. Waste water is conducted to the ground through a thin sheet-steel pipe. A vertical pipe open at both ends and having an area equal to 50% of the sectional area of the riser pipe is supported in the center of the surge tank by a spider. This device was constructed purely as an experiment, but with the idea that, when the water level in the surge tank is changed, the water level in the vertical pipe would change more rapidly than the water level in the surge tank and that some beneficial effect might be produced in the way of breaking up the surges. Observation indicates that at times the water in the vertical pipe falls as much as 18 in. below the water level in the surge tank and there is apparently a decidedly beneficial effect in the regulation of surges. The overflow pipe has a conical entrance.



LONGITUDINAL SECTION OF PENSTOCK, SURGE TANK AND OVERFLOW PIPE, BEAVER RIVER, N. Y.

Some time ago the lower portion of the vertical overflow pipe collapsed without any apparent cause, as there was no ice at the time and the foundation and supports of the overflow pipe remained secure. The overflow pipe was easily straightened and reinforced with steel rings. As winter was then approaching, the entire surge tank was housed in, thus affording opportunity to observe directly the operation of the tank and overflow. It was found that when a surge occurs sufficient to cause overflow, say with a height of 6 in. above the outlet, the conical entrance to the overflow pipe becomes entirely filled with water for an instant and this mass of water in rapidly flowing downward produces a very strong indraft of air. Surge pulses producing overflow often occur in succession at intervals of perhaps one or two seconds. Each overflow sends a definite volume of water down through the overflow pipe. These conditions afford an explanation of the collapse of the overflow pipe.

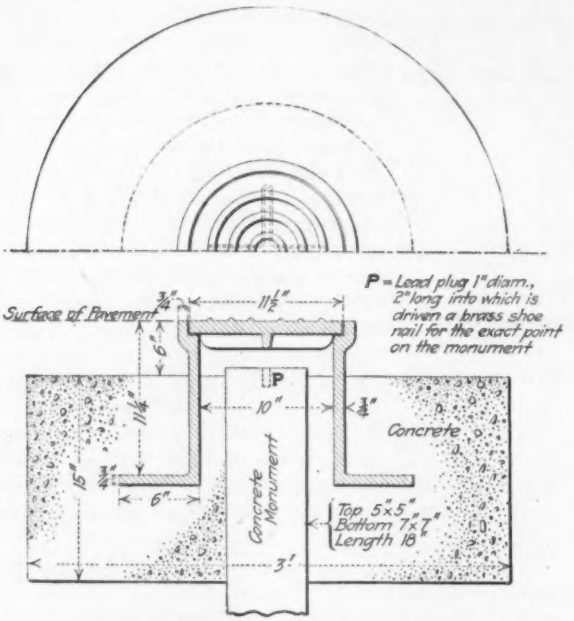
At the time of the collapse, there had probably been two successive strong pulses. The overflow from the first had reached a point probably about mid-height of the overflow pipe and was traveling downward through the overflow pipe with a velocity of 30 or 40 ft. per sec. At this instant the overflow from the secondary surge completely filled the funnel mouth, preventing the inflow of air. As the overflow from the second surge had not yet acquired any considerable downward velocity, the entrapped air was quickly rarefied by the more rapid downward flow of the lower water slug, causing sufficient difference between the external and internal atmospheric pressures to crush the lower portion of the overflow pipe before the water at mid-height had reached the bottom.

NOTES

Emergency Meals for Philadelphia Water-Works Employees—A commissary was added to the facilities of the bureau so that men suddenly called upon for all-night work can be furnished with hot coffee and food. This is not only a humane expedient, but good business policy as well, for men called unexpectedly from home, or required in an emergency to continue all night, following an ordinary day's work, cannot supply themselves with proper food. * * * During the severe snowstorm in March, when traffic on street-car and railroad lines was seriously interrupted, the operating forces at a number of the outlying pumping stations and filters were unable to leave the stations for many hours nor could the relief shifts get there. These men come to work provided with only one meal. Naturally when their shift was increased from 8 to 24 hr. the problem of securing food became urgent. To provide against such a situation in the future, arrangements have been made for placing certain standard food supplies at these outlying stations which may be drawn upon in case of such emergencies to aid the men to continue at work without exhaustion and keep the plants running.—From advance copy of 1914 report of C. E. Davis, Chief, Bureau of Water, Philadelphia.

Cinders or Broken Stone for Concrete Sidewalk Foundations?—A New England city engineer raises the query whether a cinder foundation is less liable to heave under frost action than a foundation of broken stone. His theory is that there should be no difference, but his experience is that the stone shows the most heaving. Other engineers' experiences are asked.

Center-Line Street Monuments, Pueblo, Colo.—The accompanying sketch shows a type of street monument used in Pueblo, Colo. These are of concrete, 5 in. square on top, 7 in. square at the base, and 18 in. long. The center is a lead plug 1 in. in diameter and 2 in. long, and the exact point of intersection is a brass shoe-nail driven into the lead plug. One monument is placed at each street intersection, 30 ft. from the north and 30 ft. from the east side of the street. In earth streets the monuments are placed, without protection of any kind, about 8 in. to 1 ft. beneath the surface. On paved streets a cast-iron box is placed over the top of the monument, the top of the box flush with the surface of the pavement. We think the system ideal, and it has been in use here for 30 years. We have at present 1440 of these monuments established.—D. P. Graymore, City Engineer, Pueblo, Colo.

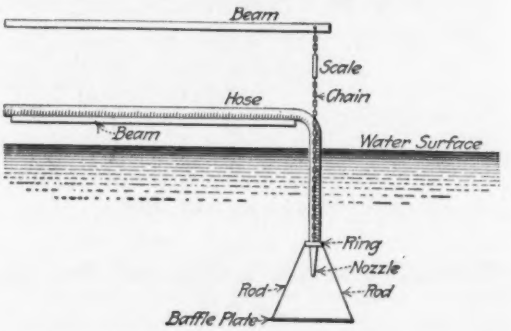


STREET MONUMENT, PUEBLO, COLO.

Lighting by Acetylene Flares for expediting construction jobs is increasing, according to a statement of the Prest-O-Lite Co., of Indianapolis, the makers of portable acetylene equipment. The following interesting cases are cited: These flares were used by J. P. Holland, of San Francisco, in the Stockton St. tunnel. Mahoney Brothers, of San Francisco, who used acetylene flares in their work on the municipal street-railway line, reported 20% greater progress than with kerosene torches. The United Iron Works, of Iola, Kan., has equipped several large dredges with these flares, and the American Contracting Co., of St. Louis, has used them on difficult dredging work. The Dunphy-Fridstein Co., of Milwaukee, used acetylene tanks fitted with long stems and several small burners in order to finish a concrete building on time; they found common lanterns insufficient and temporary electric service too expensive. The Central Union Telephone Co., of Indianapolis, for underground or night work is using small portable tanks fitted with ½-ft. burners. The unit is hung on a crossarm or any convenient projection. The Boston, Mass., and Newark, N. J., fire departments, among others, use these flares, the latter having a large special unit for alley lighting by reflection from building walls. The Underwriters' Fire Patrol, of San Francisco, uses portable units for night fires, the flares being carried on the sides of the wagons. The Benson Lumber Co., of Astoria, Ore., uses acetylene tanks with burners in ship lanterns to mark the large lumber rafts which have to be towed from the Columbia River to San Diego, Calif. The rafts are deserted for three weeks and the lamps burn continuously. The Massachusetts state militia owns several flares which last year lighted the officers' service-school encampment, covering a ground area of 400x1200 ft.

Recent Practice in Illinois State Highways—During the latter half of the year 1914 the Illinois Highway Commission entered into contract for the construction of 75 separate state-aid roads, of which 18 were vitrified brick and 57 portland cement concrete. These roads are almost invariably of either 10-ft. or 18-ft. width. The 18 brick roads aggregate 114,188 sq.yd. in 17.7 mi. and are located in 11 counties. The 57 concrete roads aggregate 539,933 sq.yd. in 75 mi. and are located in 37 counties. The surface of the brick roads is made up of 4-in. vitrified paving bricks, with a 6-in. flush concrete curb on each side. The bricks are grouted with a 1 to 1 portland cement filler and are cushioned on 1 in. of compacted sand. The foundation consists of a 1:3:5 concrete base supported upon the natural soil. The thickness of the base at the edges is 4 in., but on the center line it is 5 in. for 10-ft. roads and 6 in. for 18-ft. roads. The concrete roads are of a 1:2:3½ mixture of one-course construction. The thickness of the slab at the edges is 6 in., but on the center line it is 7 in. for 10-ft. roads and 8 in. for 18-ft. roads.—H. E. Bilger, Road Engineer, Illinois State Highway Department, Springfield, Ill.

Impact of a Submerged Water Jet—The impact of water from a jet-hose under water was measured by the apparatus shown in the accompanying sketch, on a large government piledriving work along the Mississippi River, last summer. A large iron baffle-plate fastened to an iron ring by two iron rods was suspended under the jet-hose by means of chains from the ring to a beam extending from the side of a piledriver. A spring-scale was in series with the chain. The jet-hose was supported by another extended beam. The distance of the plate from the nozzle was regulated by the chains. F. Y. Parker, U. S. Assistant Engineer, St. Louis, Mo., was in charge of the piledriving.



APPARATUS FOR MEASURING IMPACT OF WATER JET UNDER WATER

City Survey Monuments in Brisbane, Australia—In a recent survey of the City of Brisbane, Australia, for the design of a sewerage system, the streets were surveyed and marked with monuments intended to be permanent. These monuments are described as follows: "The mark used consisted of a solid concrete block 18x18 in. by 6 in. deep, with a galvanized-iron tube of 1 in. internal diameter passing through the center of the block and projecting 2 in. [?] above and clear of the block. This block was then sunk in the ground after a proper foundation had been prepared for it. The galvanized-iron tube was then protected by a cast-iron cover with a movable lid, and the excavation was filled in with stone rubble well packed and rammed and pitchers set in position, bringing the whole up to the level of the roadway. Brass cone-shaped center plugs were inserted in the tube to form a definite mark for survey purposes while the top of the tube was used as a benchmark for leveling. An iron cap was placed on the top of each tube in order to protect it from being blocked up with silt, etc. When the mark is in use the cap is removed. These marks with their protective coverings are termed Datum Boxes."—E. J. Matthews, Authorized Surveyor and Staff Surveyor to the Metropolitan Water and Sewerage Board, Brisbane, Australia.

Paper Tracings Treated with Carbon Paper as a Substitute for Ink Tracings—The use of carbon backing for paper tracings, described on p. 592 of "Engineering News," Mar. 25, is, to my personal knowledge, not less than 28 yr. old; for I made some detail drawings just 28 yr. ago for an engineer by placing black carbon paper under thin tracing paper on which the drawings were made with a lead pencil. At that time the process was not considered new, so it may be assumed the idea is as old as the idea of blueprinting. Considerable detailing is done in my office and much of it is done in this manner. I know that the majority of engineers who specialize in structural-steel detailing have been using this method for years.—Ernest McCullough, Chicago, Ill.

Editorials

The time-worn subject of rail corrugation and its causes is brought into notice again through the publication by the Institution of Civil Engineers of a paper, with an extended discussion, on the corrugation of rails, the whole forming a pamphlet of over 100 pages. It may be noted incidentally that this publication appears more than a year after the date on which the paper was presented and discussed! It cannot fairly be said, however, that this delay in publication has worked any serious injury, since the conclusion is that nobody knows what does cause the corrugation of railway rails. A great number of theories have been presented, but none has ever been accepted by the profession generally as satisfactory.

As illustrating how serious a matter rail corrugation is to the electric-railway interests, it is stated that the rail corrugations on the electric railways of the city of Glasgow necessitate an expenditure of at least $50,000 a year. It is remarkable, indeed, that a phenomenon so widespread should have remained an unsolvable puzzle to the engineers of every civilized country for so many years. Apparently no solution is likely to be reached until the problem is tackled by some large scientific organization, such as the United States Bureau of Standards, for example, equipped with sufficient funds to employ trained scientific observers to bring together and sift the data from a variety of sources and to reach a logical conclusion.

Porous Foundations at Two Dams

The Beaver Park Dam construction, described on another page of this issue by M. C. Hinderlider, should be doubly interesting to engineers on account of the remarkable duplication of geological conditions there with those at the Cedar Lake reservoir at Seattle, as noted by C. E. Fowler in ENGINEERING NEWS of Jan. 21, 1915. In both cases a nose or promontory extending into a narrow and steep gorge offered an admirable location for one of the abutments of the dam; in both cases this promontory proved to be of porous formation not at all similar to the solid rock ledge from which it apparently rose. The essential difference in the two cases is that at Cedar Lake the solidity of the promontory was assumed in the face of repeated adverse expert advice—with consequent expense and danger not now to be estimated—while at Beaver Park borings revealing the treacherous sub-base were considered, if not before work was begun, at least in time to take remedial measures before water was let into the reservoir.

The experiences with these two dams bear out, possibly in a more sensational manner than usual, the trite observation that an impervious foundation is the first requirement for a dam. This almost axiomatic principle would hardly seem to need emphasis were it not so frequently disregarded, with results varying in seriousness from mere mental stress, as at Beaver Park, to failure and heavy repair expense, as at the Stony River dam in West Virginia.

Architects' and Structural Engineers' License Laws

Besides the letters published in recent issues on the controversy over the respective rights of architects and structural engineers to erect buildings in Illinois, we have received a number of others which we have not taken space to print. There is evidently an irreconcilable difference between the opposing parties. The structural engineers claim that all they desire is the legal right to engage in the construction of buildings having no important architectural features. The architects claim that the structural engineers are seeking to obtain the sole authority over all buildings containing structural features of importance, leaving the architects in control only of buildings so small that their structural design could not be held to affect public safety.

It seems to us that this controversy in Illinois is an instructive object lesson to the profession at large as to some of the difficulties that are likely to be experienced if the project of state examination and registration for engineers should be carried into effect. Every little while, we may be sure, rivalry would appear between the engineers and the members of some other profession—the architects, for example—or between different branches of the engineering profession. Each side would engage in a struggle to secure legislation to protect its interests and defeat its opponents. The honorable profession of engineering would in such a case bear a most unpleasant resemblance to the labor organizations, which are continually exercised over the claims of rival unions as to which shall have the right to perform a piece of work.

There may be cases, as we have repeatedly stated, where certain classes of engineering work are of such importance to the public that some form of state examination and registration of those engaged in it may be required for the public protection. Where such a condition exists, legislation is proper and defensible. But legislation is not proper and will in the end react to the disadvantage of those who promote it, if its main object is to secure to a certain faction the monopoly of certain work and exclude from that work others equally competent to perform it.

Public Authority over Works for the Use of Water

The assembling this week of the convention which is to revise the Constitution of the State of New York makes it an opportune time to call attention to one change in the organic law of that state which is urgently demanded in the public interest and with which engineers are especially conversant. We refer to the need of authority to exercise the right of eminent domain in connection with a variety of works for the development and use of water. At the Constitutional Convention of 1894, Clemens Herschel, of New York City, proposed an amendment to the New York Constitution, reading as follows:

The necessary use of lands for the construction and operation of works serving to retain, exclude or convey water for

agricultural, mining, milling, domestic or sanitary purposes is hereby declared to be a public use.

It should be understood that the Constitution now prohibits the taking of private property by eminent domain except for public uses.

This proposed amendment was rejected by the convention. Instead, it adopted an amendment intended to remove a part of the difficulties in the way of constructing channels for agricultural-land drainage. When this amendment was tested in the courts, however, it was found to be of little value.

In the twenty-one years since the Constitutional Convention of 1894, great progress has been made in many parts of the country in the drainage of swamp lands and in the use of water for irrigation, public water-supply and power development. Experience of engineers in New York and in many other states has made clear the public need that the power of eminent domain should be available in the construction of many hydraulic works. Lack of that power or undue restrictions upon its use have been a great handicap in many lines of engineering work.

Mr. Herschel has renewed his recommendation to the convention which assembles this week, urging that the present importance of water development should be recognized by suitably amending the Constitution in the manner explained.

The recommendation is of interest not only to engineers in New York, but to those in many other states. It is true that in the Western states where the use of water for irrigation has become important, and in certain other states in which land drainage has become active, laws or constitutional amendments have been enacted enabling the right of eminent domain to be used in connection with works for the use of water. There are plenty of other states, however, where the old doctrine still holds that works in connection with the use of water by persons or corporations are a purely private enterprise and that lands necessary for such works, therefore, must be purchased by private agreement. This, of course, enables a single owner of land to block the entire enterprise.

Permitting and Limiting Public-Utility Competition

Since the recent decision of the California Supreme Court sustaining the State Railroad Commission in the case of the Ora Corporation it seems to have been commonly believed in some quarters that a policy of this state has now been fixed preventing all local competition between two similar utilities. While this decision does temporarily prevent a particular company from selling electric service in the City of Stockton,* yet it does not indicate the whole policy of the California Commission. The real position of the Commission was thus succinctly stated three years ago (Pacific Gas & Electric Co. vs. Great Western Power Co.):

> Where a territory is served by a utility which has pioneered * * * and is rendering efficient and cheap service * * * and the territory is so generally served that it may be said to have reached saturation as regards the particular commodity, then certainly the design of the law is that the utility shall be protected within such field; but when any one of these conditions is lacking the public convenience may often be served by allowing competition to come in. * * *
>
> If any territory served by an existing utility is afflicted with excessive rates or inefficient service and a second utility of the same kind desires to enter such territory, and the Commission should say to the existing utility, "Although when you had matters your own way you lost sight of your public duty, yet we will still preserve for you this territory in consideration for your future good behavior," in how many instances would a new utility apply when the only effect of its expense and trouble would be the cheapening of sales and improvement of service of the existing utilities? * * * We would in effect say to all the offending utilities: "You may proceed with your present methods until competition knocks at the door of your territory and only then will you be compelled to do justice." We would be saying to every new public utility: "You will knock in vain at the door of any field now served." * * *
>
> We announce the rule that only until the time of threatened competition shall the existing utility be allowed to put itself in such a position that this Commission may find that the patrons are adequately served at reasonable prices.

The actual situation the country over as to permitting competition in utilities is somewhat parallel to the regard for water rights in various states. For instance, in California the need to appropriate and permanently divert water is recognized under some conditions, but the superiority of the usual riparian rights of river-bank owners prevails where these enter a case. California's attitude is thus midway between that of the arid states where the doctrine of appropriation has full sway and that of the Eastern states where only riparian right have to be considered. The California attitude on utility competition is a middle-ground stand. In contrast the reports of the Massachusetts Gas and Electric Light Commissioners show many cases indicating that competition will be generally denied. (Although in Fitchburg a large transmission company was allowed to sell to large customers whom the original local company could not have handled). The other eastern commissions tend to follow such denials as long as existing utilities properly perform their service, and none of them seem to feel that they need to use competition as a club. Probably they see other ways of insuring good service at fair prices.

The Screw-Spike Vindicated

The screw-spike as a rail fastening has been in extensive use on European railways for many years, and attempts have been made at various times to introduce it into the United States. Half a dozen years ago a particularly vigorous effort of this sort was made, the results of which went far to sustain the contention of those who declare that the average railway official never sees any merit in a device or appliance which is different from that to which he is accustomed. Various authorities allege that trials of the screw-spike in service showed that its theoretical advantages were not borne out in practice, and that while it might be all right for European tracks it would not do for American railways.

In view of the numerous reports as to difficulties experienced with the screw-spike, it is of especial interest to note a paper in the March *Bulletin* of the American Railway Engineering Association by G. J. Ray, Chief Engineer of the Delaware, Lackawanna & Western R.R., in which is reported the result of five years' experience in

*The Ora Corporation obtained a franchise from the City of Stockton and applied to the Railroad Commission for permission to operate in competition with the Western States Gas & Electric Co. The commission passed an order in favor of the Ora company for territory outside Stockton, but for the city and certain adjacent territory the right was refused. The Ora company had both orders annulled in the Supreme Court, but on rehearing the Commission was upheld. The questions narrowed down to a consideration of whether or not a certificate of convenience was a prerequisite to the right of the company to engage in business in Stockton. The court held that the city did not have power to grant these franchises when the public-utilities act was passed, so that the requirement of a certificate did not impair any power of local control vested in the city.

the use of screw-spikes on that system. Mr. Ray states that in the last five years the Lackawanna has used 5,120,000 flat-bottom tie-plates and about 12,272,000 screw-spikes. His opinion of the screw-spike may be summed up in the following quotation:

> As a whole, the screw-spike installation has proven very satisfactory, and no conditions have developed which would cause any doubt about the ultimate success of the undertaking.

Mr. Ray makes no claim that the screw-spike is the final word in rail-holding devices. He distinctly intimates that it may some time be necessary to adopt some type of heavy bolt for securing the rail to the tie on lines of heaviest traffic. He declares, however, that five years' experience on the Lackawanna fully demonstrates the superiority of the screw-spike to the cut spike, both from the point of view of safety and as a means of reducing injury to the tie.

The standard screw-spike used by the Lackawanna is 6 in. long from the under side of the head to the end of the point and 7/8 in. in diameter under the head. Holes are bored in the tie for the screw-spike; and it has in fact been found advantageous to bore holes for cut spikes.

It will be noted that tie-plates and screw-spikes are applied together, and that the Lackawanna has adopted flat-bottom tie-plates, it having been found that the various types of flanged tie-plates which were supposed to be necessary to cut into the tie and prevent all motion between the tie-plate and the tie generally cause decay to set in under the tie-plate.

The standard Lackawanna tie-plate is 10 5/8 in. long, 7 in. wide and 3/4 in. thick, with raised lugs on the upper side of the plate around the holes for the screw-spikes and a substantial shoulder against which the outer flange of the rail bears. With these plates properly secured by screw-spikes no difficulty has been found with wear between the tie-plate and the tie, nor has it been found that the tie-plates make a noisy track.

It is admitted that the screw-spikes cost more to purchase and to apply than the ordinary cut spike. On about 60 miles of Lackawanna track laid in New Jersey with screw-spikes the cost of boring the ties and applying the spikes is given by Mr. Ray as $320 per mile. Under these conditions, as he says, the question whether or not it will pay to use screw-spikes will depend on the cost of ties and their probable life and the amount of traffic to be carried.

The New York Telephone Rate Revision

The change of the telephone-rate system in New York City just announced, as noted elsewhere in this issue, is spectacular because of a total reduction in annual gross revenue amounting to three million dollars, because hundreds of thousands of contracts will be rewritten and because a large volume of traffic that has formerly been handled on a toll basis now becomes local under the new rates. The last public proposition of the telephone company aimed to return 8 per cent. on the value of property used in rendering the service; but the service demanded by the Public Service Commission has been considerably increased so that the company will probably earn that rate only by the new economies and improvements it may be able to effect in the three short years granted by the Commission before further demands may be made.

The case is significant because it takes a long step toward the wide establishment of telephone rates on a zone-service and zone-rate basis. The telephone facilities of the country have generally been in the hands of broad-minded men with high ideals of public service; and, taking each concern's business as a whole, the returns up to now have not apparently been more than warranted by business risks. The returns in certain limited sections of a company's territory have been more than what are usually recognized as fair rates, but in other sections returns have been correspondingly reduced. There is merit in the contention that the value of urban service is increased by promoting service beyond the municipal boundaries and that city rates should help make this possible. In the earlier history of telephone service, rate making was largely by intuition and experience. In recent years, however, the principles have become better defined, and the necessary departures from elusive "costs of service" demanded by experience and judgment have become more widely recognized. However, no system of cost accounting yet devised has shown the specific cost of the various subdivisions of the general telephone service without arbitrary assumptions and apportionment. Even if such a system could be devised it would be of limited service, since for a long time to come more general satisfaction of the subscribers in the various classes will probably result with telephone rates fixed more by value of service rendered than by cost.

There is one very significant difference between this case and the first thoroughgoing telephone-rate revision, that in Massachusetts in 1910 (see ENGINEERING NEWS, Oct. 20 and Dec. 15, 1910). The substitution of a zone-service system in Boston was worked out by a commission's engineers and accountants, though with the coöperation of the company. In the present New York case the neighborhood-zone system and the metered-service rates were worked out by the company and accepted with a few modifications by the Commission.

The most striking thing about the new schedule, perhaps, is that the entire service contemplated under it is metered and charged for according to a sliding scale above a minimum. The second point in importance is the division of the metropolitan area into zones with local-call privileges within and between adjacent zones and 5-cent or 10-cent tolls to the more distant sections.

It is of particular interest to students of municipal affairs to note the judicial attitude of the Commission in the matter of apartment-house rates. In New York City the apartment-house owners form a strong group and they have loudly insisted on such rates for private-branch-exchange service as would allow them to pay their telephone bills by charging five cents a call to their tenants. The Commission apparently was not impressed with the justice of the demands made by the real-estate men, it being of the opinion as a result of its investigations that if the apartment-house owner would assign a proper value to his interior use of the telephone system (for which he pays nothing) and to the benefit in making his apartments attractive and of high rental values, he would still be able under the new charges to furnish his tenants with the service at five cents per call.

The ability of this Commission to secure great concessions for the public which it represents, and yet to shape its orders according to the dictates of evidence rather than the pressure of interested opinion, will greatly encourage every believer in the possibilities of state regulation.

Letters to the Editor

Tie-Rods for Floor Arches

Sir—Referring to the article by Frank N. Kneas in your issue of Mar. 18, and your editorial comment on the same, regarding the spacing of tie-rods for floor arches, I should like to quote the following from my book, "Steel and Reinforced Concrete in Buildings" (1911), p. 121:

> Tie-rods in hollow-tile construction are very often inadequate, as a little calculation will show. Take, for example, a 10-in. arch on a 6-ft. span. Assuming a total load of 150 lb. per sq.ft. and an effective depth of the arch of 6 in., the thrust per foot of the arch is found to be 1350 lb. It is common to see ¾-in. tie-rods spaced 6 or 8 ft. apart in beams of this size. At 6 ft. the stress on a rod is 8100 lb., or about 33,000 lb. per sq.in. on the rod at root of threads. Besides this heavy stress on the rod the side force on the flange of the beam is too great, and rods should be closer to stiffen the flange. In addition to this the rods are very often placed at the middle of the web of the beam, whereas they should be near the bottom flange, say about 3 in. from the bottom of the beam.
>
> It is true that the thrust of one arch will balance that of the next one, but this does not apply to the last arch of a row, where a single tie-rod is often made to take the full thrust of the floor arch.
>
> A good practice, and one that ought to be generally adopted, is to double the number of tie-rods for all outer arches whether these are adjacent to a stairwell or other opening in the floor or against a wall.

There is certainly need of reform as suggested by Mr. Kneas in spacing tie-rods, but the full strength of rods in interior panels is scarcely needed. The tile arch is not flexible horizontally as his Fig. 2 would indicate. It could stand a very heavy side thrust if its own arch thrust were taken care of.

Mr. Kneas shows his tie-rods at about the middle of the depth of the I-beams. They should be close to the bottom flange so as to increase the effective depth.

EDWARD GODFREY.

Pittsburgh, Pa., March 20, 1915.

Computing Area of Segments of Circles

Sir—In ENGINEERING NEWS of Oct. 29, 1914, there appeared an article by William Goldsmith, on "Snow Disposal Experiments in Manhattan Borough Sewers." Mr. Goldsmith made some calculations on the discharge through circular sewers, in which he used an approximate formula, two-thirds of the middle ordinate multiplied by the chord, for determining the area of cross-section of the following sewage. This formula is considerably in error for certain cases, and is over 3½% in error in the example he has worked; not, of course, a serious error in the computations in which he has used it.

I am inclosing a table computed for segments of circles having a radius of one, showing the errors involved in two approximate methods of calculation. The formulas used are:

1. *Area* = ⅔ *middle ordinate* × *chord*
2. *Area* = $E/15\ (8\,D + 6\,C)$

The second formula, as shown in the table, is very nearly correct, being only 1.1% in error for the special case where the chord is also the diameter of the circle.

The table may be used to compute areas of segments of circles having any radius, by entering the table with the argument, ratio of B to A, and multiplying the area of segment of a circle of 1-ft. radius by the square of the radius of the given circle; or the area may be computed by the approximate method, and increased by the proper percentage as shown.

H. N. BRADSTREET,
U. S. Transitman.

Phœnix, Ariz., Feb. 19, 1915.

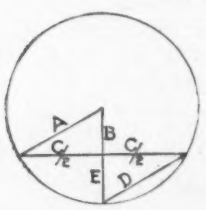
A Graphic Method of Determining Runoff to Sewers

Sir—In his reply to my criticism of his article on "A Graphic Method of Determining Runoff in Storm Sewers," O. Hufeland brings up the matter of the Additive Method presented by the writer to the Municipal Engineers of New York in 1909.* While Mr. Hufeland states that he is quite familiar with the method, he makes some remarks regarding it which prove the contrary. He says it is an effort to prescribe a theory based on few data resulting in a theoretical rainfall curve and a slide rule for sewer computations, based more or less on this curve.

My sewer slide rule has not the slightest connection with the Additive Method, nor with any rainfall curve. It is based on Kutter's formula and is equally convenient for making sewer computations by the Additive Method, the Rational Method, the Graphic Method, or the old empirical formulas. It was especially designed, however, to readily compute the time of concentration

*Mr. Hufeland's article was published in "Engineering News" of Nov. 5, 1914. Letters on that article by Mr. Nordell and others, with Mr. Hufeland's reply to them, appeared in our issue of Dec. 31, 1914, pp. 1322-5. Mr. Nordell's "Additive Method" paper was reprinted in our issue of Mar. 11, 1909, pp. 265-8.—EDITOR.

DETERMINATION OF ERRORS IN AREA OF SEGMENT DUE TO APPROXIMATE METHODS OF COMPUTATION



Radius A	Chord Distance B, Ratio, B to A	Chord Length C	Chord Length D	Methods of Computation — Approximate Area — True Area	Formula 1	Formula 2	Per Cent. to be Added to Approximate Area to Get True Area — Formula 1	Formula 2
1.0	0.0	2.000	1.414	1.5708	1.3333	1.5540	17.81	1.10
1.0	0.1	1.990	1.342	1.3712	1.1940	1.3605	14.84	0.78
1.0	0.2	1.960	1.265	1.1735	1.0451	1.1670	12.29	0.55
1.0	0.3	1.908	1.183	0.9800	0.8904	0.9759	10.06	0.42
1.0	0.4	1.833	1.095	0.7926	0.7332	0.7903	8.10	0.29
1.0	0.5	1.732	1.000	0.6142	0.5774	0.6131	6.37	0.18
1.0	0.6	1.600	0.894	0.4472	0.4267	0.4467	4.81	0.11
1.0	0.7	1.428	0.775	0.2956	0.2857	0.2953	3.48	0.10
1.0	0.8	1.200	0.632	0.1635	0.1600	0.1634	2.19	0.06
1.0	0.9	0.872	0.447	0.0586	0.0581	0.0587	0.86	....

to help the man who believes that the time element is a useful factor in sewer design.

I am not financially interested in the sale of the rule, and, aside from a desire to hasten the general adoption of the rational methods of design by furnishing a means of greatly reducing the labor involved in the calculations, I have no interest in pushing its sale.

As to the Additive Method being founded on few data, it is founded on no data at all, in the sense used by Mr. Hufeland. It is a mathematical method, capable of mathematical demonstration and proof. I take this to be true of the Rational Method and the Graphic Method. In Metcalf and Eddy's book on "American Sewerage Practice," the basic principle of the Rational Method is spoken of as being axiomatic—that is, self evident.

The place where experimental data come in, in any of the above methods, is in the choice of the hypothetical storm to design to, and the fundamental assumptions made.

In the use of each of the methods certain assumptions are made and certain conventions used in order that their application may be practicable. Of the conventions used, certain ones are common to all three methods. All use a runoff coefficient, arrived at in the same way, and all disregard the effect of storage in the mains and upon the surface. There is another factor which I believe has not as yet ever been considered. It is the phenomenon opposite in effect to storage and which in some work now underway I have termed "overrunning."

These conditions affect all three methods alike, and their discussion, as Mr. Hufeland observes, "has nothing to do with the case." The point of difference in the three methods is in the fundamental assumptions made.

The Rational Method affirms that the runoff is equal to the average rate of rainfall during the period of concentration, multiplied by the impervious area. This can be true only under one of two conditions. Either the rate of rainfall must be constant throughout the period or the collection must be uniform. The latter statement means that equal areas shall be drained in equal times. For instance, one-quarter the total area in one-quarter the total time, one-half in one-half the time, etc.

This may be realized by assuming the figures for a very simple case. Suppose that the time of concentration of a drainage basin were one hour and the effective area A. Let us also suppose that the rainfall were at the rates of 5 in. per hr. for the first half hour and 1 in. per hr. for the remainder of the time. The average rate of rainfall for the hour would then be 3 in. per hr. Now imagine that three-quarters of the drainage basin is drained in one-half the time; the concentration period for that portion, one-half hour, the rate of rainfall 5 in. per hr.; consequently the runoff $= \frac{3}{4} A \times 5 = 3\frac{3}{4} A$. Now this rate of runoff, $3\frac{3}{4} A$, passes through the remainder of the drainage basin and has added to it the runoff from that portion. As the rainfall is then 1 in. per hr. and the area drained $\frac{1}{4} A$, the runoff so added is $\frac{1}{4} A$, making the total runoff 4 A. But according to the Rational Method the runoff would be 3 A, a discrepancy of 1 A, due to the theoretical deficiency of the Rational Method when a variable rainfall is combined with non-uniform collection.

From a study of the rainfall records of New York City prior to 1908 it appeared to me that the assumption of a constant rate of rainfall throughout a storm was entirely unwarranted. The assumption of uniform collection comes closer to actual conditions but in many cases there is considerable departure from this theoretical condition.

Accordingly, I brought out the Additive Method to deal with these conditions. The Additive Method affirms that the runoff is equal to the average rate of rainfall during the period of concentration, multiplied by the impervious area, *only* for small areas, and is not true for the drainage area as a whole.

This method is thought by some to involve great difficulty in the choice of a storm to design to. This can be quite easily obviated by assuming that the rates of rainfall, as ordinarily used for the Rational Method, are all found in one storm. These rates, which are average rates for the period, determine the arrangement of intensities, in the hypothetical storm modeled upon those actually observed. This storm would then be used as a basis of design.

Where the collection is uniform the Additive Method and the Rational Method will give identical results with the same rainfall curve. It is only when the collection is decidedly nonuniform that any great variation in result appears.

I think that the Additive Method results, in such cases, in a better proportioning of the parts of a sewerage system, which is the paramount feature of design today. The economic size of the system as a whole is a matter of judgment and the engineer is handicapped in making his final decision by having few real data to go by. Neither the number of hours of flooding to be expected, with any particular design, nor the resulting damage in dollars and cents can be accurately foretold.

However, the proper proportioning of the members of a storm-water system is of greater importance than the determination of total capacity, and it is in this direction that I believe immediate progress will be made. The excess capacity of a system somewhat too large is not a dead loss, for the extra capacity will at times be utilized. A system too small performs some useful work, and the damages resulting from flooding may be partly met by the interest on the money not expended in building a larger system. But money spent in building laterals too large for the trunk sewers, or *vice-versa*, is absolutely wasted, for the capacity can never be utilized.

This is the most serious defect of the exponential formulas, outweighing even the ambiguity and uncertainty attached to the factors R and S. The nature of the formulas precludes a harmonious relation between the members of a system.

The Graphic Method, as advanced by Mr. Hufeland, is based on a constant rate of rainfall throughout the storm. It is also tacitly assumed that the critical storm is of much less duration than the period of concentration. To me it seems that the first assumption is so far from the truth that it renders the whole method invalid. A storm that commences abruptly, continues at a constant rate for a time, and then stops as abruptly as it began, is too highly conventionalized to be practicable.

The second assumption is an even graver defect. It seems to me that R. D. Goodrich's* letter, the tone of which is entirely friendly, really upsets the practicability of the Graphic Method. Mr. Goodrich suggests, and

*See "Engineering News," Dec. 31, 1914, pp. 1322-5.

rightly, that the effect of storms of different duration would have to be tried out. If this is true wherein would the Graphic Method and the Rational differ? And if the results do not differ why use the more cumbersome Graphic Method? The only argument is that a storm of short duration might possibly be the critical one for the trunk sewer. But I agree with W. W. Horner that the chance of this is very remote and could occur only with a peculiarly shaped drainage area. Even so, the Graphic Method could not be accepted as the criterion, as its basic assumption is too highly conventionalized to allow of any such pretension to precision.

Mr. Hufeland apparently thinks I am too prejudiced in favor of the Additive Method to give credit to any other method. This is not the case—I should be glad to see it "suffer from desuetude" in face of simpler and better methods. I would speedily drop both it and the Rational Method could I hit on something better, but I do not believe in making the subject any more complex by "importing" foreign methods that do not improve matters.

CARL H. NORDELL, Engineer of Design,
Sewerage Commission.

Milwaukee, Wis., Feb. 10, 1915.

[Mr. Hufeland submits the following reply to Mr. Nordell's letter.—EDITOR.]

Sir—It would require a longer letter than Mr. Nordell's to answer the arguments he advances, and I fear the total result would not add much to the sum of knowledge helpful to sewer construction, because I believe his assumptions are based on an erroneous use of rainfall data.

I base this belief on an analytical study of the records of a self-recording rain-gage in this city, covering 45 years, which has convinced me that the method of averaging universally used in the construction of rainfall curves is misleading, at least so far as records taken in New York City can be used as a guide. And Mr. Nordell uses such averages.

While the water from a "whole" storm must be carried away, it is, in my opinion, not the whole storm but a particular high intensity of precipitation at a certain portion of it that tests the capacity of the sewers, and for this the sewers should be designed. While the intensity directly preceding and following the peak load may affect it to some degree, it becomes a question to what extent it is worth considering. I well know that even putting such a question is heresy in the eyes of most practitioners, to say nothing of denying the importance of these quantities, but the study of the charts of many of the heaviest storms, and only such need consideration, has led me to believe that the peak load put on the sewers in such storms is so great, and sufficiently defined, that the quantities preceding and following this maximum rate can be neglected. The records show that, compared to the duration of the whole storm, these maximum rates are of short duration, but they are long enough to be considered as a measure to be used in sewer design. These high rates rarely exceed twenty minutes and are, in a great majority of the records I have examined, a feature of such prominence that they overshadow the periods of lesser rates. If this statement is true, and it is based on facts and not on theory, averages covering much longer periods of greatly varying rainfall must be misleading, and sewers so designed are open to criticism.

The graphic method described by me does not depend for its value on the views expressed above, but if these views are correct, and I believe they are, perhaps Mr. Nordell might concede the graphic method to be of some value. I have prepared a paper in which I have given both the "average" and the uniform-intensity methods of tabulating the records mentioned above. The publication of the paper will give advocates of both methods a chance for comparison, and perhaps for a settlement of this vexed question.

O. HUFELAND.

New York City, Mar. 10, 1915.

NOTES AND QUERIES

In the article on the substructure of the Jackson St. Bridge, Chicago, in "Engineering News" of Mar. 18, it should have been stated that the Strauss Bascule Bridge Co., of Chicago, is consulting engineer for both the substructure and superstructure work.

Mill-Building Columns.—In the discussion of how the flexibility of the truss changes the distribution of wind reactions in a mill-building bent, in our issue of March 25, 1915, p. 590, the terms n′ and n were used to express the flexibilities of truss and column. In the drawing these terms are erroneously defined (in Fig. 2). They should have been given as follows:

$$\frac{I'}{I} = \frac{1}{n'} \qquad \frac{I}{h} = \frac{1}{n}$$

When the truss is assumed perfectly rigid, I′ is infinitely large, and its flexibility n′ becomes zero; then the formula derived for flexible truss becomes equal to that derived for rigid truss.

One Contractor's Method for Holding Good Men

BY MORTON C. TUTTLE*

This describes an attempt by a company operating as general contractors to give more regular employment to its men. The scheme used is outlined with the hope that others may try something similar to soften, in some measure, the hardships caused by the irregularity of employment which has been the principal evil to both employer and employee in the building trade.

After 15 years of successful progress from small beginnings, the Aberthaw Construction Co. found that, as is the case with every other company in the building business, certain men followed its work and looked to it for regular employment. Even on its scattered work, men preferred to travel in order to stay with the superintendents they knew rather than to stand their chances of irregular employment near home.

The principal difficulty which the men experienced came from the fact that wages vary considerably in different localities. There is usually a going rate of wage for a given occupation and these zones of varying wage are surprisingly close together, owing to the fact that in the building trade most of the work in a given place is handled by the local contractors, who recognize a wage established in that section. We found that our men who had traveled naturally went to the jobs where the local wage was the highest, and we found that before a job was completed they tended to drift toward another job that was just starting or had several months to run.

In an effort to correct this condition, we issued "Record

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*Secretary, Aberthaw Construction Co., 8 Beacon St., Boston, Mass.

of Service Books" to a limited number of the best of our men. We guaranteed a book holder an agreed wage per hour wherever he worked for this company, and we agreed to give any book holder employment as soon as the company had a position available which he was capable of filling and we kept all bookmen informed of new work, and whenever new work came up we communicated with any of them who did not happen then to be employed by us.

When it was proposed to do this there was a feeling among our superintendents that bookmen were likely to neglect their work on the feeling that they had assured positions with the company. It was pointed out that the system of signing in and out on a job would prevent this, and each superintendent had the right to refuse to sign out any unsatisfactory man. As a matter of fact, the assurance of regular employment tends to make a man more faithful and easier to handle than is the floater.

The method of signing in and out with these books is as follows: When a man presents his book on a new job, the superintendent fills in the left-hand page, stating that he began work at a given date on a named job. At the end of the job, if the man's work had been satisfactory, the superintendent signs the statement that he quit work on a given date, and sends his book to the main office, where a sticker is attached to the right-hand page, and this is signed by an officer of the company, showing that the man has served properly on this job and that his book is good. The same process is repeated on future jobs. The book not only serves to give this man regular employment with our company, but is a first-class recommendation to any other company who happens to value our opinion in regard to workmen in our line. In order that the book shall not be transferable, it contains at the front a very brief description of the holder, giving nationality, age, height, weight, color of eyes, color of hair, and his signature. A carbon copy of this page is torn out and held at the main office.

In 1910 an investigation was made as to the average earnings of carpenters and form builders in our employ. About the only information that could be obtained was from the men themselves, and this being a matter of memory was naturally incomplete and unreliable. The opinion was formed, however, that the average earning of the best men, who were paid 40 to 50c. per hr., was between $15 and $18 per week, seldom reaching the higher figure in the course of a year, owing to lack of work, interruptions on account of storms, and other delays incident to outdoor employment on an hour basis.

In February, March and April of 1911 we issued books to 21 men, and in the remainder of that year we issued three other books. In 1912 we issued 23 books, in 1913, six books, in 1914 five more—a total of 68.

The following table gives the number of weeks that each one of these men worked for this company with the maximum and minimum earnings per week:

TABLE SHOWING WORK OF BOOKMEN FOR YEAR 1914

Total number of men holding books		68
Canceled for inefficiency or for not being reliable	2	
Number of book holders moved to foreign countries or engaged in other occupations	9	
	—	11
Active book holders		57
Number of book holders away 1 to 6 mo. of their own accord and not available for work	14	
Number of book holders losing work due to inefficiency or for not being reliable	4	
	—	18
Number of men available for full year's work		39

Average time of employment of each of these 39 bookmen	50 wk.
Smallest employed period	40 wk.
Average yearly earnings	$1388.90
Average weekly earnings	27.76
Highest yearly earnings	1736.99
Lowest yearly earnings	886.92

These figures show that these men have earned wages that compare favorably with wages of men in less seasonable occupations, and indicate the possibility of making profitable an occupation as irregular as that of the building trade.

The figures on the earnings of the various men show advantage to the employees. From the point of view of the employer, we can state that the result has been satisfactory to us, that we have probably obtained better return for our money from these men than from any others that we have employed. One has only to consider the extraordinary expense of teaching an inexperienced man a new trade to realize the fearful waste that there is in the building trade in picking up unorganized gangs and hiring, firing, and teaching, until a reasonable degree of efficiency has been obtained. The cost of an average reinforced-concrete building is made up of about 35% labor and 65% material. More serious than the inefficiency in the labor, itself, is the destruction and the waste of material. The employer's power of control is necessarily limited where a man expects for the rest of his life to drift from one job to another.

The gradation of punishments in the building trade is very crude, there usually being two electives—to call a man down severely, to which the man usually does not object, or to fire him, which in many cases makes very little more impression on him because he expects to be out of work a large part of the time and looks on such a discharge as simply meeting one of the evil days that he expects to meet frequently. With regularity of employment, proper and humane discipline can be brought into the building trade. The Record of Service Books used by this company offer an opportunity for this. For some misdemeanor or other a bookman who has completed his work may be given his book not signed by the superintendent, and yet the latter may not consider the matter serious enough to cancel the book altogether. This omission puts the man on the same basis as a man without a book except that if he can get located again and properly signed out he becomes a book holder again in good standing. The lay-off of graded length, the threat of discharge or lay-off, and the criticism of work in which a man takes pride, all become more effective when the employee is with an organization offering him steady employment.

Road Work in Kentucky—The General Assembly of Kentucky in 1914 increased, the state tax from 50c. to 55c. per $100; the extra 5c. is to go toward state road building, and it is estimated this yearly road fund together with the motor vehicle tax will be $600,000. This sum is to be apportioned to the several counties applying for state aid and agreeing to pay half the cost of road building. About $1,500,000 will probably be expended on state-aid roads this year. In addition to this work five counties have voted bonds aggregating $1,500,000, which will be spent under the state department of highways. Thirteen other counties are preparing to vote on bonds aggregating $2,015,000, so that prospects of road builders are very bright in Kentucky. The state legislature has outlined a system of roads for the state, which consists of roads connecting the county seats throughout the state by the most direct and practicable routes. These roads will be the first to receive state aid, and after such roads have been constructed in all the counties, then all other roads which may be agreed upon between the state and county authorities are to be improved.

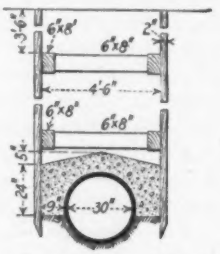
Reinforcing a Vitrified-Pipe Sewer at Passaic, N. J.

By S. J. Naughton*

The city of Passaic, N. J., has just repaired a large vitrified-pipe sewer by reinforcing it with concrete.

During the year 1899 the city constructed a 30-in. storm sewer in Hope Ave. between Sherman St. and Van Buren St. In 1901 this sewer was extended south to Monroe St. The bed on which the sewer was constructed was very wet sand and some difficulties were encountered.

In 1912 work was commenced on Section 15 of the Main Auxiliary Sewer of the Passaic Valley Sewerage District. This section of the sewer runs through Hope Ave., 16 ft. c. to c., east of and parallel with the 30-in. storm-water sewer. The district sewer at this point is 7½ ft. in diameter, which brought the sheeting of the trenches close to the vitrified-pipe sewer. The construction of this big sewer caused the bed of the 30-in. sewer to slide or settle. This was unavoidable on account of the two sewers being so close and the larger one much deeper than the smaller. When the sand at the sides and bottom of the small pipe settled it brought a heavier pressure on the vitrified pipe than it could stand. The pipe cracked on top and at the sides, a number of the joints spread apart, and some sections (two and three lengths in some places) settled. These cracks were caused by pressure alone and not by defective pipe.



Pipe Sewer, Passaic, N. J.

The laying of a sheet-asphalt pavement was under consideration by the Board of Commissioners of Passaic; but before doing so it was thought necessary and advisable to repair the worst sections of the sewer. In order to ascertain the extent of the damages done by settlement and other causes an interior inspection of the sewer was necessary. On Dec. 2, 1914, City Engineer C. R. Wise and one of his assistants, R. F. Odell, made a trip of inspection through the sewer, examining carefully each length of pipe and every joint for 850 ft. The covers of the manholes at each end of the section under inspection having been removed, the writer was stationed in one of these manholes to take down notes as to the conditions found by the inspector as he went through. This work was made uncomfortable by the stench of sewer gas and by small pools of water in places where the sewer had settled, at times 3 and 4 in. deep. The inspection showed that the sewer would have to be repaired and made safe against further settlement before the pavement was laid.

The accompanying sketch illustrates the method by which the pipe was reinforced. A trench 4½ ft. wide was dug, 2-in. spruce sheeting was driven, and 6x8-in. rangers and cross-braces were put in. The pipe was then stripped to a point 4 in. below the springing line and 1:3:6 concrete placed around it and well rammed. When the concrete had set the sheeting was cut off 3 ft. below the surface of the street and allowed to remain in place with the rangers and braces. These acted as a form for the concrete and kept a uniform pressure on the top of the pipe. Some of the pipe, it was found, was not cracked, but the joints had spread apart. These were calked and sealed from the inside with cement mortar. The trench was then backfilled and well puddled with water, thus insuring against settlement after the pavement is laid.

*Bureau of Engineering, Passaic, N. J.

Reducing the Cost of Brick Roads in Illinois*

By H. E. Bilger†

If it were possible to reduce the first cost of vitrified-brick pavement construction by some 30%, the adoption of this type of pavement for Illinois roadwork in the near future would be limited only by the capacity of the brick plants to furnish the material.

By providing a little more for engineering service, a more detailed study of the local conditions can be made, to the end that the requirements of the specifications may be better adjusted to meet the varied physical conditions encountered.

Without a detailed study of the actual conditions of each particular piece of work, with the arbitrary requirement of such dimensions and materials as will most provide so called fool-proof construction, expenditures for highway improvement cannot be conducted along the most economical lines. It would seem that by the present revision of our specifications for brick pavement construction, such economy will be brought about as will promote a somewhat more general adoption of this type of pavement.

With due consideration having been given to the subgrade, there appears no necessity for the concrete base having other than a uniform 4-in. thickness for 10-ft. pavements, and 4½ in. for 18-ft. pavements. For brick pavements of no greater width than 18 ft., the practicable and economical means of providing for the pavement crown is by crowning the subgrade, rather than by increasing the thickness of the concrete base at the center.

It is not to be expected that subgrade conditions will be uniform throughout the length of any pavement construction job, and consequently the close study of details during construction, as well as while making the survey, will enable exceptional conditions to be provided for by increasing the thickness of the base, or by some other means.

It is believed that a concrete mixture of 1:3½:6 for the base will in the end prove more economical than the 1:3:5 mixture which has heretofore been used. In the light of our requirements as to the fine and the coarse aggregate, the practice of specifying a very rich concrete throughout the entire length of the pavement, in order to care for possible weak spots in the subgrade, is not economical. All isolated weak spots in the subgrade should be treated as individual matters, and should be taken care of by special treatment.

With proper and practicable workmanship in striking off the surface of the concrete base, there appears no reason why the sand cushion after compaction should have a thickness of more than 1 in. A greater thickness of sand cushion, and failure to compact it are conditions that permit the sand to enter the spaces between the bricks and thereby seriously reduce the efficiency of the grout filler.

With respect to the bricks themselves, it would seem that

*From a paper read before the Illinois Society of Engineers and Surveyors.

†Road Engineer, Illinois State Highway Department, Springfield, Ill.

the first opportunity for lowering the construction cost would concern their quality rather than their size. For average rural traffic requirements, it is not always economical to require the hardest bricks commercially available for this purpose. A condition that makes for a serviceable brick pavement is not the absolute hardness of the brick but rather the uniformity of quality.

For most all rural traffic conditions in Illinois, bricks that are satisfactory in other respects should provide a serviceable and economical pavement, providing the loss by abrasion does not exceed an average of 22% and on any individual brick 26%.

There is every reason to believe that without noticeably reducing the quality of the pavement, our revised specifications will permit the construction of brick pavements at a figure some 15 or 20c. per sq.yd. lower than the prices paid in 1914. However, it is not apparent that under the conditions in question, the first cost of the brick pavement can be reduced materially below $1.80 per sq.yd.

Revision of New York City Telephone Rates

The New York Telephone Co. about a month ago proposed to the Public Service Commission a new schedule of rates and a new basis of service for New York City. Since then there has been a series of conferences at which the demands of the Commission and the counter propositions of the company have been considered. The Commission, under date of Mar. 30, accepted the plan outlined below and ordered it to be effective for three years from July 1.

The Commission has agreed that a return of 8% on a valuation of $84,000,000 for the company's plant should be one aim of the rate schedule. There are about 300,000 subscribers and 560,000 telephones in New York City.

The new plan divides Greater New York into 10 zones, as shown in the accompanying map. Subscribers' calls inside one zone and between adjacent zones are treated as local traffic; beyond that they are subject to toll charges of 5 or 10c. The local rate from public telephones remains 5c. and the toll rate 10c. to other city points. This basis is best shown by the following table:

NEW YORK CITY LOCAL AND TOLL RATES

Zones	1	2	3	4	5	6	7	8	9	10
1	L									
2	L	L								
3	L	L	L							
4	L	L	5	L						
5	L	5	10	L	L					
6	5	5	5	L	L	L				
7	5	5	10	L	L	L	L			
8	5	10	10	5	L	5	L	L		
9	5	10	10	5	5	10	10	10	L	
10	10	10	10	10	10	10	10	10	L	L

The schedule of rates for various classes of local service (L in the above table) is based on metering the system* and charging in accordance with a sliding scale above minimum charges, as indicated in the typical abbreviated tabulation below. Flat-rate contracts (those which were not obsolete under the old system) may be retained, although in general it is expected that the new scheme will be much more attractive where calls are made out of very restricted neighborhoods.

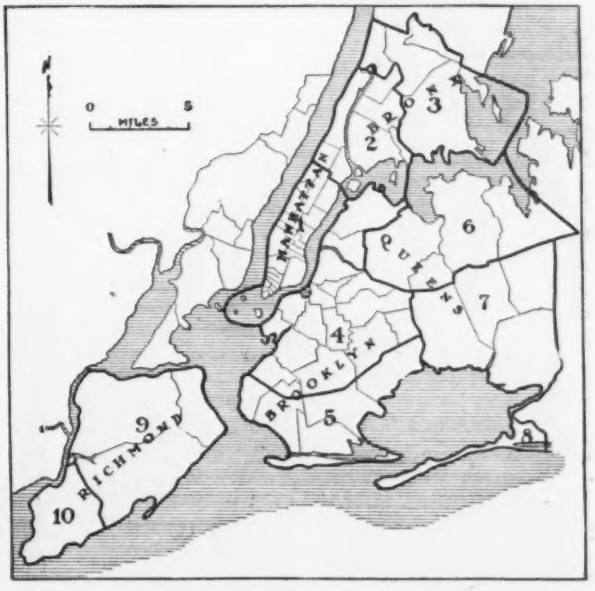
*All the single-party lines in New York City are already equipped with meters. These are only partly automatic, as their control is intentionally left to the operators, since "connections" are not always "conversations," and all fire, police, ambulance and emergency calls are free.

TYPICAL LOCAL TELEPHONE RATES, NEW YORK CITY

Local Messages Zones 4 and 5	Individual Line	Two-Party Line	Four-Party Line
600	..	..	$30
720	..	$36	36
800	$40	40	40
1020	51	51	51
2100	96	..	..
3300	135	..	..
3600	144	..	..
4800	174	..	..

It is seen that these rates are based on charges starting at 5c. per call and decreasing to 2½c., compared with 8 to 3c. before.

The rates of Zones 1 and 2 are similar to those shown, except that there are no four-party lines in the latter. Two- and four-party lines are provided for by the new rates in Zones 2, 4 and 5, but not in 3, 6, 7, 8, 9 and 10. The last six zones are outlying and have considerable num-



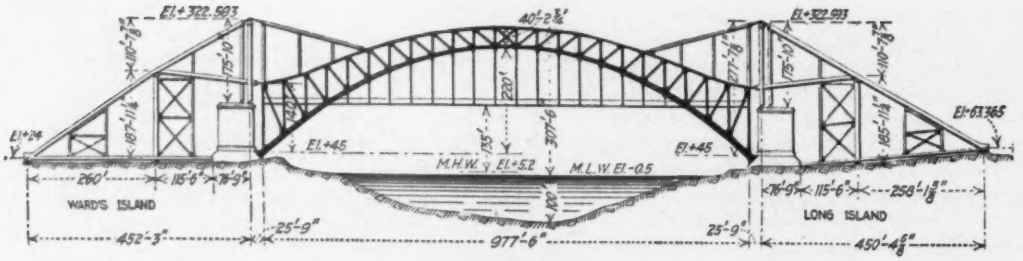
TELEPHONE ZONES IN NEW YORK

bers of small subscribers, so that this arrangement seems peculiar. It should be explained that it is intended to place all business service throughout the city on a meter basis, but that in Zones 3, 6, 7, 8, 9 and 10, two- and four-party residence lines are to remain in restricted-neighborhood flat-rate unmetered service at from $4 to $6 less than the minimum figures in the above schedules.

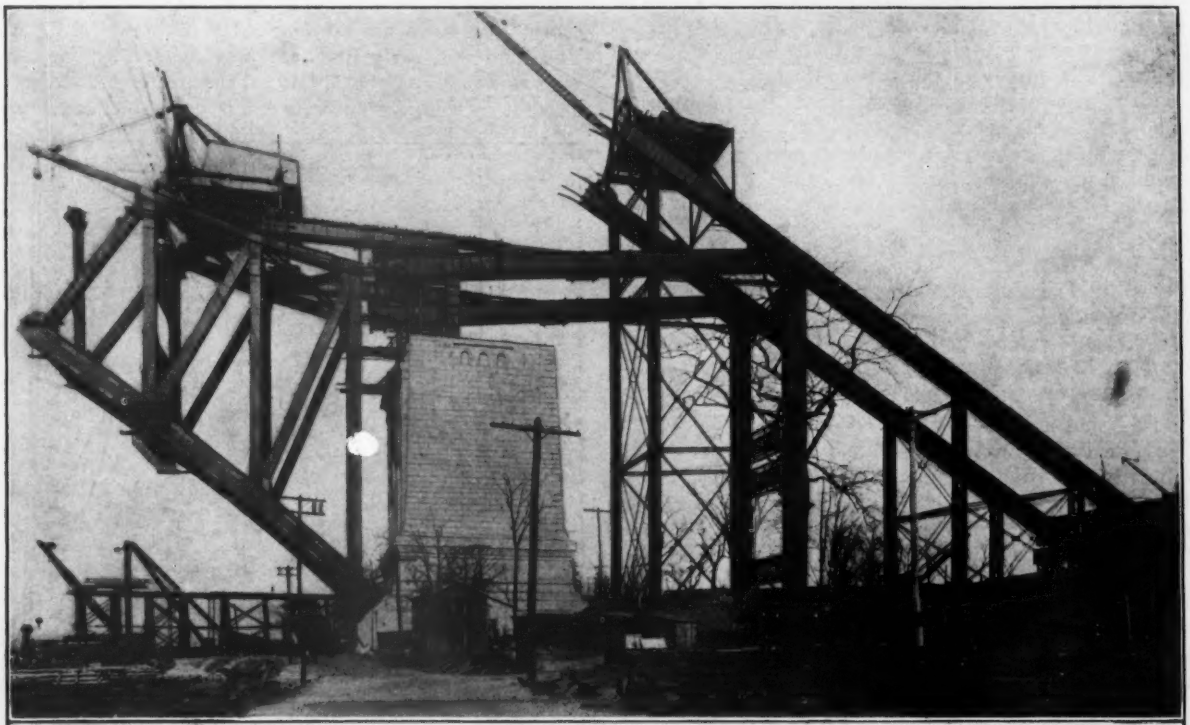
The rates in Zones 3, 6, 7 and 8 (for individual lines only) run from $36 per year for a 720-message contract to $135 for 3600 messages. For Zones 9 and 10 the rates are $36 to $112.50 on the same basis.

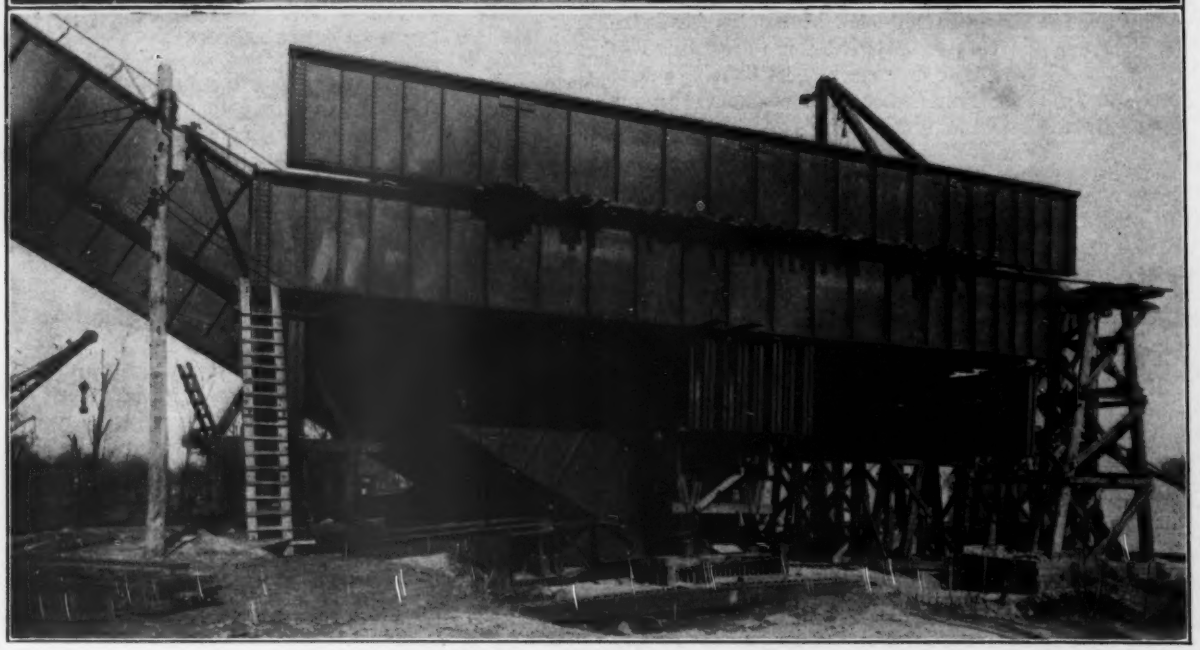
Pressure has long been exerted, particularly by apartment-house owners, for the reduction of prices of private-branch exchange service so that the bills might be met by charging tenants 5c. per call. The following rates allowed by the Commission make an initial reduction from about 6 to 5½c. For Zones 9 and 10 the costs of switchboards, with service, are $120 and $126 instead of $126 and $132 elsewhere.

Monitor boards, two stations, one trunk line and 2400 local calls per year	$126.00
Cord switchboard with the same service	132.00
First 10 extra stations, each	6.00
Second 10 extra stations, each	4.80
Above 20 extra stations, each	3.60
Extra switchboard drops above 30, each	1.20
Extra trunks, each	24.00
Extra local messages	0.025



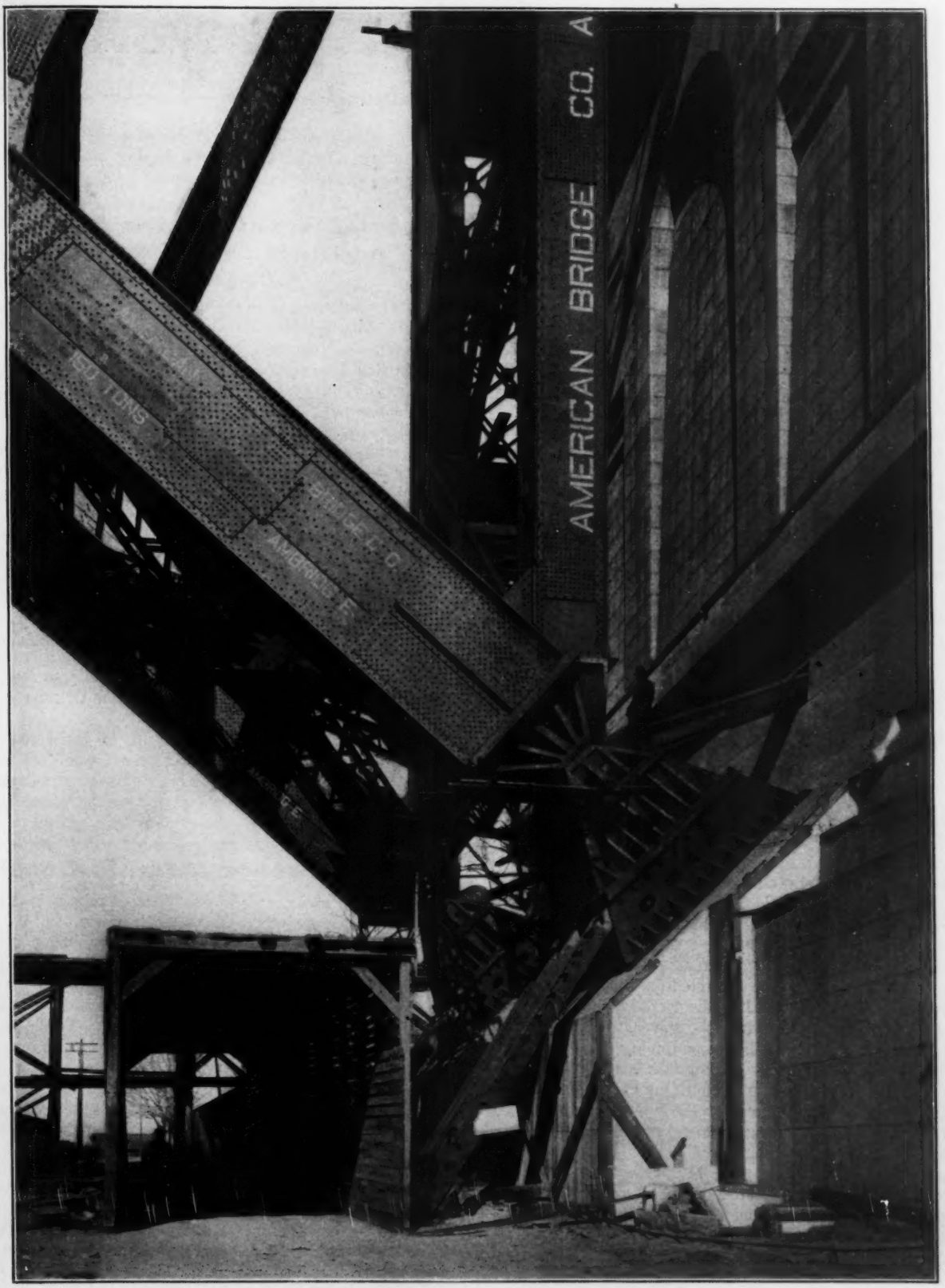
Sketch of erection scheme for Hell Gate arch (reproduced from "Engineering News," Jan. 8, 1914, p. 62). The Long Island end, at right, is the one shown in the pictures below, and in the large view opposite.





THE HELL GATE ARCH ACROSS THE EAST RIVER

(Upper view: East end of arch, and erection backstay. Lower view: Counterweight grillage at end of backstay triangle)



THE HELL GATE ARCH ACROSS THE EAST RIVER
(Detail view of end section of arch rib)

News of the Engineering World

Erection of Hell Gate Arch Progressing

Steel erection on the Hell Gate arch bridge (New York Connecting R.R.) over the East River at New York began two months ago. Rapid progress has been made on the east bank; three panels are completed at present (see views on pp. 698-9). On the west bank (Wards Island) the temporary backstay and anchorage are not yet completed. The backstays (upper, or tension, member and lower, or compression, member) are made largely of stringers of the arch span, while the anchorage uses some 1400 tons of girders belonging to the viaduct spans of the approach.

The main traveler handled the 165-ton end sections of the bottom chord without difficulty, hoisting them from landing-cars to which the yard derrick, below, transferred them from the railway cars on car-floats at the dock front.

The backstay will shortly be carried up to full height, with support on a post resting on the top of the abutment tower, and with a connection thence forward to the fourth panel-point of the top chord. To transfer the anchorage pull from the lower backstay to the upper, two jacks of 2400 tons' capacity under the saddle of the upper backstay will lift this enough to allow disconnecting the link of the lower backstay.

Wisconsin State Building Code in Practice

Progress is being made in Wisconsin under the building code drawn up last year by the Industrial Commission in coöperation with building and contracting interests. The code is general and is intended to be supplemented locally by additional ordinances. Its enforcement is in the hands of the local authorities. Since the adoption of the code in October state officials have been doing educational work among local officials and councils with a view to securing the preparation and adoption of local supplementary ordinances. A suggested building ordinance for small cities was prepared by the commission to assist this work. In a number of cities the mayor, fire chief or other officer has taken interest in enforcing the state code. Where the local officials do not take hold of the matter adequately the supervision of building operations by deputies of the commission will probably be tried. Architects and owners have to a large extent made use of the code, and have submitted plans to the State Commission for approval.

A bulletin on fire protection in schools has been prepared and distributed, and a similar one is in preparation to cover factories. Requirements for fire protection in existing buildings have been prepared in tentative form, and will probably be adopted in a few months. Requirements for light, air and other sanitary provisions in tenements are also to be issued by the commission.

The general course of events is held to constitute an approval of the idea of a state code having minimum requirements and informational character.

Special Trains to the International Engineering Congress

A joint committee representing the American Societies of Civil, Mining, Mechanical, Electrical, and Marine Engineers, has arranged for a special train from New York to San Francisco for the convenience of engineers desiring to attend the International Engineering Congress, in September next, enabling them to travel across the continent together.

The route selected is by the New York Central Lines and the Santa Fé System. The train will leave the Grand Central Terminal in New York City at 7:45 p.m. Thursday, Sept. 9, and is due in San Francisco on the following Wednesday at 9 p.m. The itinerary includes a half day's stop at Niagara Falls, a full day at Colorado Springs (giving time to visit the summit of Pike's Peak or the Garden of the Gods), and 15 hours at the Grand Cañon. The round-trip rate for the railway ticket, which is good for 90 days to return over any desired route, is $98.80. The one-way Pullman rate is $22 additional.

The committee has also arranged for special cars from New Orleans to San Francisco, leaving New Orleans on Sunday, Sept. 12.

Circulars giving full information as to the accommodations, etc., on these trains and hotel accommodations in San Francisco in connection with the Engineering Congress may be obtained by applying to the secretary of any of the national engineering societies.

Blizzard Sweeps Atlantic Coast

A snowstorm accompanied by a 30- to 60-mile gale did considerable damage along the Atlantic Coast, on Apr. 3. The snowfall, by far the greatest of the winter on the Atlantic Coast, ranged from 10 to 15 in. High tides at Norfolk, Va., caused the inundation of some of the streets, while telephone and telegraph communication and railway and ferry service in the city were paralyzed. Traffic on the surface and elevated lines in New York was demoralized, and railway transportation was impeded to some extent. Four transatlantic liners and a dozen other steamships, which were to have sailed on Saturday, were held up on account of the storm. Several coastwise ships were lost in the gales.

Record Drought at Boston

Another record drought at Boston ended on Apr. 3, when after 36 days of no measurable precipitation a heavy snowstorm covered the ground. March is usually a wet month, and never before since records have been kept has it passed without rain or snow. The drought, although

not of so long duration as the one of September and October, 1914, when only 0.21 in. of rain fell in 47 days, has been a serious one because of the low stage of many water-storage reservoirs. The nearest approach to this record was in 1874, when there were 25 days without precipitation in August and September.

United States Railways in 1914

Statistics of railways in the United States for the year ending June 30, 1914, were made public by the Interstate Commerce Commission on Mar. 31. The mileage of railways in the United States on June 30, 1914, was 247,400 miles of line and 377,102 miles of track. The increase in trackage during the year was 7522 miles. The rolling-stock equipment of these railways includes 64,760 locomotives, 53,466 passenger cars, 2,325,647 freight cars and 124,700 cars in company service. These figures do not include cars owned by other than railway companies. These railways are operated by 1,695,000 employees, who received in wages and salaries during the year $1,373,422,000.

The total capitalization of these railways was $20,247,000,000, of which $8,680,000,000 is stock and the remainder bonds and other funded debt. The average rate of dividend on all stocks issued was 5.20%. No interest was paid on 12% of the funded debt. The average receipts per passenger mile were 1.982c. The average receipts from freight per ton-mile were 0.733c. The earnings from freight business were about three times as great as the earnings from passenger business. Of the operating expenses of $2,200,000,000, $419,000,000 was for maintenance of way and structures, $532,000,000 for maintenance of equipment and $1,165,000,000 for traffic and transportation.

Novel Iron-Removal Plant

An iron-removal plant for Mt. Auburn Cemetery, near Boston, is now being constructed to remove iron from driven-well water used to sprinkle the grave lots. To prevent staining of gravestones the cemetery authorities were compelled either to cover the gravestones when the driven-well water was used for sprinkling or else to buy water from Cambridge, Mass., for sprinkling purposes. The iron-removal plant consists of a pipe-and-nozzle aërator, a trickling filter of coke, a settling reservoir and sand filter beds operated at a high rate. The plant is designed to treat 1,000,000 gal. in 24 hr. and is practically like the iron-removal plant at Middleboro, Mass. (see ENGINEERING NEWS, Feb. 26, 1914, p. 458), except that the Mt. Auburn plant has a coke bed 5 ft. deep and the Middleboro plant has one 10 ft. deep. Like the Middleboro plant, the one for the Mt. Auburn Cemetery was designed by Robert Spurr Weston, Consulting Engineer, 14 Beacon St., Boston.

A Gas Explosion in the carbolic-acid plant of the Semet-Solvay Co., Syracuse, N. Y., on Mar. 31, injured 18 persons and shattered the building.

An Unusual Railway Accident—Many car windows were broken, on April 3, by snow wedged between an eastbound Long Island R.R. train at Hicksville, L. I., and a rotary snow-plow which was clearing the westbound track. Several passengers were cut by flying glass. The train, consisting of an engine and four cars, continued to Greenport.

The Guarantee of New Pavements required of contractors by the City of Syracuse, N. Y., has been reduced to extend over 5 years instead of 10, as has been the practice hitherto. The new ordinance establishing the 5-year guarantee period for all types of pavement was advocated by the mayor and the paving committee of the local chamber of commerce.

Draining Swamps of Brazoria County, Texas.—Drainage work is going forward on the Clemens State Farm, Brazoria County, Texas. Four surveying parties are in the field. The work started some months ago, but was suspended on account of floods. State Reclamation Engineer Arthur A. Styles estimates that the work, which involves the reclamation of 36 sq.mi., will be completed within six weeks. Convict labor furnished by the state is being used.

Drainage for Irrigated Lands is the subject of a bulletin just issued by the Department of Agriculture. It is stated that more than a tenth of the irrigated lands in the United States which have been irrigated for some time are now either absolutely unproductive or are used only for poor pasturage and unprofitable crops. Irrigated land when not drained is likely to be ruined either by water-logging or the accumulation of harmful mineral salts.

The Canadian Patent on Bitulithic Pavement No. 88,116 was sustained by Justice Hyndman of the Supreme Court of Alberta in a suit brought for its infringement by the City of Calgary, which let a contract for 100,000 sq.yd. of pavement to be identical with a pavement which the Bitulithic & Contracting, Ltd., had laid in the same city in 1910-11. The Canadian patent in suit was said by the Court to be practically identical with the U. S. Patent No. 727,505 and was adjudged valid and infringed by the defendants.

Competition with the State-Owned Universities is not confined to the United States. In a recent issue of "The Canadian Engineer" there appears the following "Personal":

W. H. Day, B. A., professor of physics, Ontario Agricultural College, reminds "The Canadian Engineer" that for a number of years the College has been making drainage surveys for farmers free of charge, except for traveling expenses. This offer is again renewed. Farmers having drainage difficulties may secure assistance by writing the department of physics, Ontario Agricultural College, Guelph, for information and regular application forms.

The Safety Improvements of the New York Subway (Interborough company's system) recommended by Alfred Craven, Chief Engineer of the Public Service Commission, as noted in "Engineering News," Feb. 4, 1915, have been ordered by the commission. These include a separate telephone system with emergency boxes between stations, independent lighting circuit with outside supply, independent supply of power for ventilating fans, rearrangement of power, signal and lighting cables and improvement of emergency exits.

New York Barge Canal Work completed to Jan. 1, 1915, is summarized in the "Barge Canal Bulletin." Including the work in the Cayuga and Seneca canal, the aggregate quantity performed to that date includes in round figures 100,800,000 cu.yd. excavation, 10,700,000 cu.yd. embankment, 2,700,000 cu.yd. concrete, 1,500,000 cu.yd. rough stone, 25,000,000 ft. b.m. sawed lumber, 2,350,000 lin.ft. piling and round timber, 91,000,000 lb. metalwork at pound prices, and $2,600,000 worth of metal paid for by the piece, besides over $3,000,000 worth of miscellaneous work.

A New Automobile Speedway, which promises to become one of the most notable in this country, has been begun in Brooklyn, N. Y. This is the famous Sheepshead Bay racetrack, which has been purchased by a syndicate for $2,400,000. It is proposed to spend $1,000,000 in transforming the old horse-racing track into a modern motor speedway, with a polo field, baseball and football fields, tennis courts and aviation field in the enclosure, which includes about 430 acres. The two-mile racecourse is to be built on a plan similar to the one at Indianapolis. It is expected it will be ready by Sept. 6.

Prizes for Street Designs—A competition open to all engineers, architects, or students in engineering or architecture is announced by the Municipal Art Society, 119 East 19th St., New York City. Prizes of $300, $200 and $100 are offered for the best solution of the problem of designing a street and avenue intersection, with reference to the demands of congested traffic, the value of abutting property, and beauty. Drawings must be delivered to the society on May 22. The designs will be judged by a committee including the Mayor and thirteen other city officials, architects and engineers. The chairman of the society committee is William N. Taylor.

Hannibal Bridge to be Rebuilt—The old Hannibal bridge at Kansas City, used by the C., B. & Q. R.R., is to be rebuilt. The Burlington company owns a one-third interest in the new Fratt bridge (built on the old Winner bridge piers), the lower deck of which is for railway traffic. However, it has never been possible to make connections owing to the fact that there is one-half mile of right-of-way on the Kansas City side that is lacking in order to connect the bridge up with the nearest Burlington tracks. The superstructure for

the new bridge has been let to the Union Bridge & Construction Co., Kansas City, on a percentage basis (6%).

Detroit's Offer to Purchase the property of the Detroit United Ry. was accepted by the stockholders of the company on Mar. 31. The purchase now comes before the voters of the city. No cash is involved in the deal, the city assuming a mortgage debt of $24,900,000 for lines in the single-fare zone. The company owns several interurban lines, to Toledo, Ann Arbor, Jackson, Pontiac, Flint, Port Huron, etc. The total assets amount to $47,688,000; the liabilities include $12,500,000 in capital stock, $24,950,000 in mortgage bonds, $2,024,000 in depreciation funds and $5,000,000 in surplus. There are 793 mi. of track, 1550 passenger cars and 355 service cars.

The Emergency Snow Removal Forces assembled by the New York City Street Cleaning Department to fight the storm of Apr. 3 amounted to about 20,000 men, and it is estimated the work will cost the city $100,000. After 24 hours' work 14,500,000 sq.yd. of pavement had been cleaned by shoveling and pushing the snow into sewers; 3,120,000 sq.yd. had been cleaned by shoveling into piles for contractors to remove, and 1,225,000 sq.yd. was cleaned and the snow carted away by the city's own street cleaning forces. This covers the work in the three Boroughs of Manhattan, Brooklyn and the Bronx. The motor-truck snowplows, which owing to inexperience in operation were not very successful at the beginning of the season, are now among the most efficient means of snow handling.

The Canadian National Transcontinental Ry. from Moncton, N. B., to Winnipeg may be operated by the Canadian Government. In the Canadian Parliament on Mar. 31, Hon. Frank Cochrane, Minister of Railways, stated that he was advised by the Chief Engineer of the National Transcontinental that the road was now complete for operation. The government preferred that the Grand Trunk Pacific should take over the line and operate it under its contract, but as the Grand Trunk Pacific may decide not to take advantage of its right to lease the road, the government desires authority to operate the road itself in connection with the Intercolonial Railway System, which is already operated by the government. Parliament has appropriated $5,000,000 for the National Transcontinental Ry. and $5,500,000 for the Hudson Bay Ry.

The Standard Steam Boiler Specifications formulated by a committee of the American Society of Mechanical Engineers have been approved by a joint committee representing the American Boiler Manufacturers Association and the National Tubular Boiler Association. This action insures that the standard boiler code will have behind it practically the entire boiler-making industry. A number of leading boiler manufacturers announced at the meeting that in the future all their boilers will be built in conformity with the standard code without regard to whether it is required by state legislation or not. The representative of a leading boiler-inspection company also announced that its specifications would hereafter be drawn in conformity to the standard code. Copies of the code can be obtained from the American Society of Mechanical Engineers, 29 W. 39th St., New York City.

Conference of Mayors and Municipal Exhibit—A municipal exhibit, the first of its kind in New York State, will be one of the features of the sixth annual meeting of the New York State Conference of Mayors and Other City Officials to be held at Troy, June 1, 2 and 3. The exhibit will consist of implements and manufactured wares most commonly used and purchased by municipalities. Mayor Cornelius F. Burns, of Troy, has appointed a local committee to take charge of the exhibit and act for the conference. The exhibit will be held in the large drill shed of the Armory. Plans are being made so that the large apparatus may be easily moved to the street for demonstration purposes. The Conference is planning to make the exhibit attractive, not only to the visiting officials, but also to the people living in the ten nearby cities connected with Troy by trolley. All of the cities in the state have been asked to send motion pictures of their principal municipal activities. C. R. Metgzer, general secretary of the Troy Chamber of Commerce, is secretary of the committee in charge.

Exhibition Runs of the Eight-Car Steel Trains of the New York Municipal Ry. Corpn. for the new Manhattan-Brooklyn rapid-transit subway system were held last week. The design and equipment of this new rolling stock were noted in "Engineering News," Oct. 15, 1915, p. 785. Briefly reviewed, the cars are 67 ft. long, 9 ft. 9⅞ in. wide, have three doors in each side under the control (electro-pneumatic) of a trainman in the center of the car, have seats for 78 passengers and standing room for 72, and weigh 85,000 lb. Every car in a train is a motor car (with two 140-hp. motors). Electro-pneumatic brakes with clasp shoes are installed and the head and tail lights automatically show the proper color. All the air, electric, safety-gate and chain connections, formerly made by hand, are automatically made by the couplers. The floors are of magnesite cement, and all stanchions, rails and grips are finished in vitreous enamel. The ceilings and upper sides are white to increase the illumination, which is furnished by fifteen 50-cp. tungsten lamps in shades. There is also a storage-battery emergency lighting system. The seats were designed by posture experts. There are 24 automatic roof ventilators.

Fuel for Steamers at Panama—By Apr. 1 the storage capacity for fuel oil at Balboa, which is the principal point of loading on the Panama Canal, were 220,000 bbl., and 37,000 bbl. of Diesel-engine oil, according to a recent issue of the "Canal Record." All of the tanks containing the oil are connected with the government oil-crib through the handling plant of the Panama Canal. The depth of water along the crib is 45 ft. at mean tide. Plans have been approved for the building of two additional cribs when the demand shall warrant it. Under the present arrangement oil can be delivered to vessels at rates up to about 2000 bbl. per hr. The fuel-oil storage capacity of Cristobal at present is 195,000 bbl., using two 42,000-bbl. tanks and one 37,000-bbl. tank. Diesel-engine oil is not now sold at the Atlantic entrance. The oil is delivered at docks 13 and 14, which have 30 ft. of water alongside. Coal can be supplied to vessels at both Cristobal and Balboa at the rate of 600 to 750 tons per day. In February, 56 vessels were supplied with 17,672½ tons of coal by the Panama R.R. at Colon. Of these, all but six passed through the canal. At Balboa the sales were only to vessels not using the canal and amounted to about 3000 tons. The preponderance of sales at the Atlantic entrance is due to the price of coal being $1 less per ton there than at Balboa.

PERSONALS

Mr. Edward M. Bigelow, M. Am. Soc. C. E., has resigned as State Highway Commissioner of Pennsylvania.

Mr. Samuel D. Foster, M. Am. Soc. C. E., has resigned as Chief Engineer of the Pennsylvania State Highway Department.

Mr. G. E. Johnson, former City Engineer of Falls City, Neb., has been appointed State Engineer of Nebraska to succeed Mr. D. D. Price, Jun. Am. Soc. C. E.

Mr. T. J. Wyche, Chief Engineer of the Western Pacific Ry., has been appointed Assistant General Manager, with headquarters at San Francisco, Calif.

Mr. Hurlbut S. Jacoby, formerly Chief Engineer of the Standard Steel Construction Co. Ltd., at Welland, Ont., has recently been appointed Instructor in Civil Engineering at Pennsylvania State College.

Mr. L. C. Walling, who has been City Engineer of Wynot, Neb., since the installation of a lighting plant in that city last June, has resigned to accept the superintendency of the electric-light plant in Hartington, Neb. His resignation will take effect Apr. 30.

Mr. H. M. Sharp, a resident of Williams Co., Ohio, has been appointed First Deputy of Construction by Mr. Clinton Cowen, newly appointed State Highway Commissioner. Mr. Sharp is a graduate in civil engineering of Armour Institute of Technology, and was formerly County Surveyor of William County.

Dr. Henry M. Payne, Consulting Engineer, 233 Broadway, New York City, left Apr. 5 for the Yukon Territory. He expects to return to the Pacific Coast in July and go to Japan and into the Lena River goldfield in Siberia to examine the mining operations there and recommend the introduction of modern equipment.

Mr. William H. Boeh, who has been Chief Deputy in the County Surveyor's office since 1907, has been appointed County Surveyor of Hamilton Co., Ohio. His first public position was that of Sewerage Engineer of Cincinnati. In 1885 he was appointed Superintendent of Water-Works, of the city, which position he held until 1906.

Prof. H. J. Thorkelson, M. Am. Soc. M. E., of the Faculty of the College of Engineering of the University of Wisconsin and Chief Bridge Engineer of the Wisconsin Highway Commission, was elected Business Manager of the University of Wisconsin by the Board of Regents on Mar. 27. Prof. Thorkelson has been Acting Business Manager for six months, and succeeds Dr. H. C. Bumpus, who resigned to become President of Tufts College. The salary is $5000 per year.

Mr. Alexander P. Gest, Special Agent of the New Jersey division of the Pennsylvania R.R., has been appointed Secretary of the Association of Transportation Officers of the Pennsylvania Lines East of Pittsburgh, Penn. Mr. Gest was born at Philadelphia in 1853. He completed his education at

the University of Pennsylvania and the Rensselær Polytechnic Institute, where he was graduated in 1874. He entered the service of the Pennsylvania R.R., in 1875, in the office of the Chief Engineer.

Mr. Alexander Robertson, Assistant to the President of the Missouri Pacific Ry., the St. Louis, Iron Mountain & Southern Ry., the Denver & Rio Grande R.R. and the Western Pacific Ry., will, until further notice, it is announced by President Bush, assume the duties heretofore performed by Mr. E. J. Pearson, M. Am. Soc. C. E., Vice-President in Charge of Operation of the Missouri Pacific Ry. Mr. Robertson was born at Albany, N. Y., in 1860, and began railray work in 1885 with the Fitchburg R.R. His headquarters will be in St. Louis, Mo.

Prof. A. E. Chandler, of the Engineering Faculty of the University of California, has been appointed a member of the new California State Water Commission by Governor Johnson The new commission will have jurisdiction over all water rights, irrigation problems and applications to appropriate water for all purposes. The former commission, which consisted of Prof. Charles D. Marx, President, Am. Soc. C. E., of Stanford University, and Messrs. S. C. Graham, of Los Angeles, and Harold T. Power, of Auburn, automatically ceased to exist with the establishment of the new commission.

Messrs. M. S. Averill, H. B. Matthews and associates have opened an office at 917 Essex Bldg., Newark, N. J., under the firm name of Averill-Matthews Co., to carry on a general contracting business, including sewage disposal and water-works. Mr. Averill, President and General Manager, has been associated for the past six years with Mr. John W. Heller, a contractor, of Newark. Mr. Matthews has been engaged in engineering and construction work for a number of years in the United States Reclamation Service, with private irrigation companies in the West, and, more recently, with the Pearson Engineering Corporation in Spain.

Mr. Joseph T. Richards, whose retirement as Consulting Engineer of Maintenance-of-Way of the Pennsylvania R.R. was noted in "Engineering News," Feb. 25, has been placed on the company's "Roll of Honor." He was largely influential, during the many years he was at the head of the Maintenance-of-Way Department, in establishing the present high standards of the company for track, stations, bridges and operating yards. Mr. Richards was born near Rising Sun, Md., in 1845, and was educated at the West Nottingham Academy, in Cecil Co., Md. He entered the service of the Pennsylvania R.R. in 1869 as rodman and transitman.

Mr. Clinton Cowen, whose appointment as State Highway Commissioner of Ohio was noted in our issue of Mar. 25, was born in 1862, and was educated at the public schools and Dennison University in Ohio. After leaving college, he devoted a couple of years to surveying and modeling and mapping the historic earthworks, ancient remains of cliff and valley dwellers in the southwestern part of the United States, after which he opened an office in Cincinnati and carried on a miscellaneous practice for a number of years. He was elected County Surveyor of Hamilton County in 1901 and has continued in the same position up to the present time.

OBITUARY

William E. Maher, a well-known contractor of Philadelphia, Penn., died recently at the Presbyterian Hospital in that city. Mr. Maher was Acting Chief of Highways under the régime of Mayor Weaver.

James McCleman, founder of the contracting firm of H. A. McCleman & Sons, of Philadelphia, Penn., died recently at St. Anges Hospital in that city as the result of injuries sustained in a fall several weeks ago. Mr. McCleman was 65 years of age.

J. S. Hill, formerly an officer of the U. S. Coast and Geodetic Survey, died recently. His principal work in the survey was the triangulation of the Texas-California arc. Mr. Hill left the survey to go into farming on a large scale in the Sacramento Valley. He was married not long before his death

Theodore Guillaudue, at one time Consulting Engineer to Thomas A. Edison, committed suicide Mar. 28 at his home in Freehold, N. J. Mr. Guillaudue was born in New York. He was a retired marine engineer and worked with Mr. Edison after retiring from active service on ocean-going steamships. He was nearly 88 years old, and left two sons and a daughter.

William Hunter, M. Am. Soc. C. E., Chief Engineer of the Philadelphia & Reading Ry., died at the Jefferson Hospital in Philadelphia, Apr. 2 as the result of a stroke of paralysis sustained while he was waiting for a train in the Reading Terminal. Mr. Hunter was born at Moselem, Berks Co., Penn., in 1854. He graduated from the Polytechnic College of Pennsylvania in 1872, and the same year entered the service of the Philadelphia & Reading as a rodman, remaining with the company for four years. Mr. Hunter then went into business for himself, and a year later entered the employ of the Pittsburgh & Lake Erie R.R. as Division Engineer. The following year, in 1878, he returned to the Philadelphia & Reading, and subsequently became Assistant Engineer, Assistant Roadmaster and Assistant Chief Engineer, being appointed Chief Engineer in 1900.

ENGINEERING SOCIETIES

COMING MEETINGS

IOWA STREET AND INTERURBAN RAILWAY ASSOCIATION.
Apr. 22-23. Annual convention at Keokuk, Iowa. Secy., H. E. Weeks, Davenport, Iowa.

AIR BRAKE ASSOCIATION.
May 4-7. Convention at Chicago. Secy., F. M. Nelis, 53 State St., Boston, Mass.

AMERICAN WATER WORKS ASSOCIATION.
May 10-14. Annual meeting in Cincinnati. Secy., J. M. Diven, 47 State St., Troy, N. Y.

NATIONAL FIRE PROTECTION ASSOCIATION.
May 11-13. Annual meeting at Hotel Astor, New York City. Secy., Franklin H. Wentworth, 87 Milk St., Boston, Mass.

CONFERENCE OF BUILDING INSPECTORS.
May 14. Meeting in New York City. Secy., Sidney J. Williams, Madison, Wis.

INTERNATIONAL RAILWAY FUEL ASSOCIATION.
May 17-20. Annual meeting at Chicago. Secy., C. G. Hall, 922 McCormick Bldg., Chicago.

SOUTHWESTERN ELECTRICAL AND GAS ASSOCIATION.
May 19. Annual convention at Galveston, Tex. Secy., H. S. Cooper, Dallas, Tex.

NATIONAL MACHINE TOOL BUILDERS' ASSOCIATION.
May 20-21. Spring meeting at Atlantic City, at Hotel Marlborough-Blenheim. Secy., Charles E. Hildreth, Worcester, Mass.

MASTER BOILER MAKERS' ASSOCIATION.
May 25-28. Annual meeting in Chicago. Secy., H. D. Vought, 95 Liberty St., New York City.

NATIONAL DISTRICT HEATING ASSOCIATION.
June 1-3. Annual convention at Hotel Sherman, Chicago. Secy., B. L. Gaskill.

AMERICAN SUPPLY AND MACHINERY MANUFACTURERS' ASSOCIATION.
June 3-4. Annual convention at Bellevue-Stratford, Philadelphia, Penn. Secy., F. D. Mitchell, 1510 Woolworth Bldg., New York City.

NATIONAL CONFERENCE ON CITY PLANNING.
June 7-9. National Conference in Detroit, Mich. Secy., Flavel Shurtleff, 19 Congress St., Boston, Mass.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 9-11. Annual convention in Atlantic City. Secy., Jos. M. Taylor, Karpen Bldg., Chicago.

MASTER CAR BUILDERS' ASSOCIATION.
June 14-16. Annual convention in Atlantic City. Secy., Jos. M. Taylor, Karpen Bldg., Chicago.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
June 17-18. Semi-annual meeting at Toledo. Secy., Frank E. Sanborn, Ohio State University, Columbus.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.
June 22-25. Annual meeting at Iowa State College, Ames, Iowa. Secy., F. L. Bishop, University of Pittsburgh, Pittsburgh, Penn.

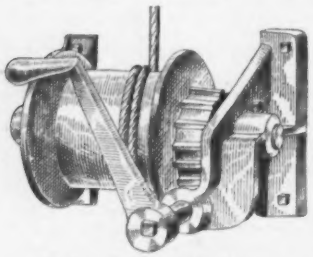
AMERICAN SOCIETY FOR TESTING MATERIALS.
June 22-26. Annual meeting in Atlantic City. Secy., Edgar Marburg, University of Pennsylvania, Philadelphia, Penn.

American Institute of Electrical Engineers—A two-day meeting will be held in Pittsburgh on April 15 and 16 under the auspices of the Pittsburgh Section and the Industrial Power Committee. The program includes the following papers: "Industrial Control in the Foundry," R. H. McLain; "Mill Controllers," H. F. Stratton; "Steel Mill Controllers," J. S. Riggs; "Control of Direct-Current Hoists," E. G. Stoltz and W. O. Lum; "Direct-Current Control for Hoists," W. T. Snyder; "Control of Alternating-Current Mill Motors," A. Simon; "Alternating-Current Hoist," R. E. Brown. The 32d Annual Convention will be held at Deer Park, Md., June 29 to July 2, with headquarters at the Deer Park Hotel, where technical sessions will be held. There will be papers on electro-physics, electrical industrial machinery, illumination and electric railroading. A special convention will be held at the Panama-Pacific Exposition in San Francisco from Sept. 16 to 18. This time has been arranged instead of attempting to hold the annual convention at about the time of the International Engineering Congress, which immediately follows the special convention. The technical sessions will be limited to the morning and afternoon of Sept. 16 and 17 in order to leave the rest of the time free for inspection trips, etc.

Appliances and Materials

Small Safety Winch

The small cast-iron hand winch for wall, post or A-frame use in small contracting work, shown in the accompanying sketch, has recently been designed by the Stevenson-Adamson Manufacturing Co., Aurora, Ill. The drum is driven by a worm and gear, so that the load is held in any position. The weight is 56 lb.; capacity, 500 lb.; drum, 6x7 in.; gear ratio, 12 : 1; and price, $8.

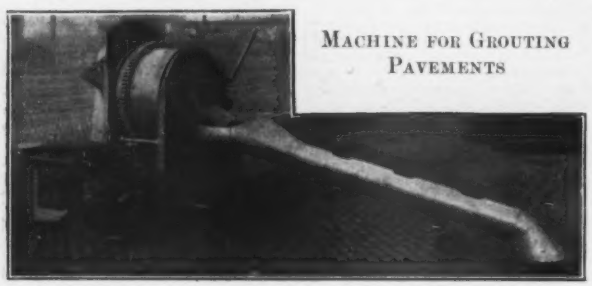


Safety Hand Winch

* * *

Grout Mixer and Distributer

A novel portable plant for mixing and distributing grout for brick paving is being put out by the Marsh-Capron Mfg. Co., of Chicago. As shown in the accompanying view, it consists of a small batch mixer mounted on a truck and discharging into the pivoted hopper end of a long pipe chute. The discharge end is fitted with a plate perforated with 1-in. holes, for distributing the grout, and is supported by a castor which allows of swinging the chute from side to side. The mixer is driven by a 2-hp. gasoline engine (not shown). The weight of the outfit complete is about 1000 lb.



Machine for Grouting Pavements

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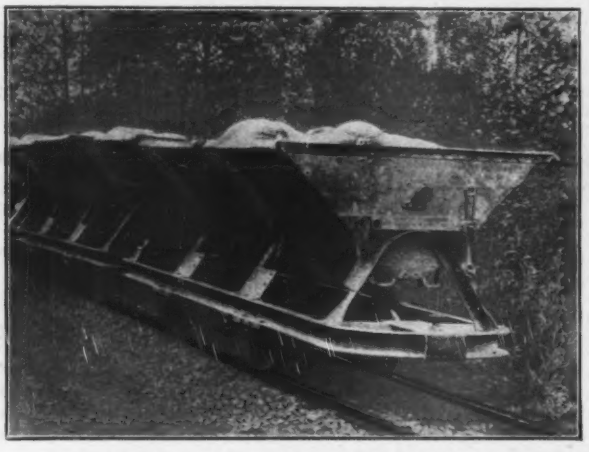
New Industrial-Railway Cars

A new type of industrial-railway car, especially designed for handling materials in concrete-road construction, has been developed by the Orenstein-Arthur Koppel Co., Pittsburgh, Penn. A steel-channel frame carries three pairs of angle-iron inverted rockers, on which rest sheet-steel bodies, or buckets, of 12-cu.ft. capacity each. Single-body cars can be made but the three-body car is more convenient, as it will carry all the ingredients of the concrete—cement, sand and stone.

The bodies, or buckets, have projecting pins, by which they may be lifted off the rockers. For this purpose concrete mixers are provided with jib cranes and traveling hoists, so that the car bodies are lifted and carried to the top of the mixer and dumped. This unit may also be used as an ordinary dump-car. The equipment shown in the accompanying illustration was built especially for the Oregon State Highway Commission.



Triple-Body Industrial-Railway Cars

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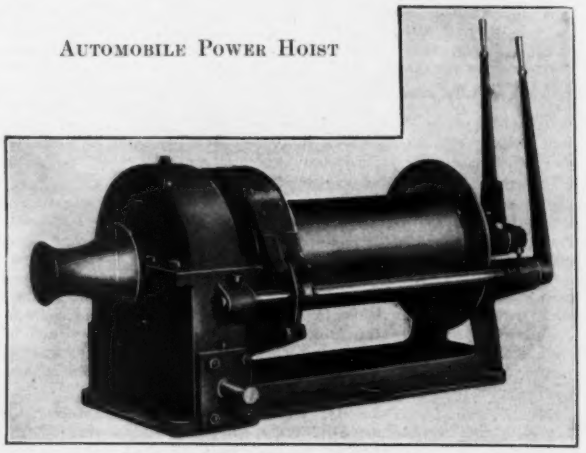
Power Winch for Trucks

The power winch for motor trucks, shown in the accompanying figure, was designed by the Northern Engineering Works, of Detroit, for one automobile concern, but has been adopted since by others.



Automobile Power Hoist

The winch is of the platform-drum type, with either spur or worm gearing, is fitted with a hand brake and is driven from a clutch connection to the truck motor-shaft. It is mounted at the forward end of the body and generally used for hauling in or out; with a frame and tackle, it can be used for hoisting. The winches are all single-line units ranging from 1000- to 2000-lb. capacity.

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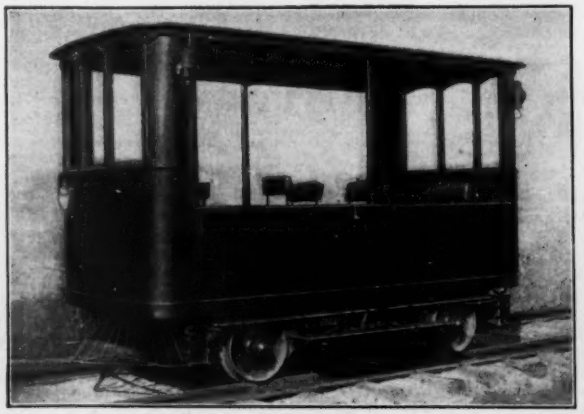
Railway Inspection Motor Car

The railway-inspection motor car shown in the accompanying view has been designed by the Sheffield Car Co., of Three Rivers, Mich., and the Fairbanks, Morse & Co., of Chicago (designated as Model 35). The car is 12 ft. 8 in. long and 5 ft. 8 in. wide as built for standard-gage track. It weighs 4000 lb.



Railway Inspection Motor Car

The car is handled like an automobile, but in construction and finish it is a railway motor car. A 30-hp. gasoline engine drives a jack-shaft through a single-plate clutch, double bevel gears and a three-speed transmission. The jack-shaft drives each rear wheel by a chain. The wheels have turned treads, spring suspension and roller bearings on a dead axle. It is claimed it can run from 4 to 50 mi. per hr. The body has a full standing roof, closed ends with low-drop windows, semi-closed sides with curtains and two doors. There are eight independent revolving seats.